Storms, Floods, and Debris Flows in Southern California and Arizona 1978 and 1980

Proceedings of a Symposium September 17-18, 1980

Sponsored jointly by:

Committee on Natural Disasters
Commission on Sociotechnical Systems
National Research Council

and the

Environmental Quality Laboratory
California Institute of Technology

National Academy Press
Washington, D.C. 1982

NOTICE: The Committee on Natural Disasters project, under which this report was prepared, was approved by the Governing Board of the National Research Council, whose members are drawn from the councils of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine. The members of the Committee responsible for the report were chosen for their special competences and with regard for appropriate balance.

This report has been reviewed by a group other than the authors according to procedures approved by a Report Review Committee consisting of members of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine.

The National Research Council was established by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purposes of furthering knowledge and of advising the federal government. The Council operates in accordance with general policies determined by the Academy under the authority of its congressional charter of 1863, which establishes the Academy as a private, nonprofit, self-governing membership corporation. The Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in the conduct of their services to the government, the public, and the scientific and engineering communities. It is administered jointly by both Academies and the Institute of Medicine. The National Academy of Engineering and the Institute of Medicine were established in 1964 and 1970, respectively, under the charter of the National Academy of Sciences.

This study was supported by the National Science Foundation under Grant No. PFR-7810631 to the National Academy of Sciences, with a subcontract to the California Institute of Technology. Other sources of support to the Environmental Quality Laboratory are listed in the preface.

Any opinions, findings, and conclusions or recommendations expressed in these proceedings are those of the authors and do not necessarily reflect the views of the National Science Foundation, the National Research Council, or the organizations of the authors.

Photograph on inside front cover courtesy of the Los Angeles Times.

Available from:

National Technical Information Service
Attention: Document Sales
5285 Port Royal Road
Springfield, Virginia 22161

Report No. CSS-CND-019
Price Codes: paper A21, mf A01

The overview and summary chapter is also available as a separate document:

Report No. CSS-CND-019A

A limited number of free copies are available on request to:

Committee on Natural Disasters
National Academy of Sciences
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

or

Environmental Quality Laboratory
California Institute of Technology
Mail Code 314-40
Pasadena, California 91125

PREFACE

The Committee on Natural Disasters of the National Research Council arranges for the timely documentation of natural disasters and their effects on man. Information on any failures of structures or systems and analyses of these failures are especially important in advancing our ability to withstand natural disasters in the future. The storms and floods of 1978 and 1980 in southern California and Arizona have been chosen by the Committee as disasters to be documented.

The Committee joined with the Environmental Quality Laboratory of the California Institute of Technology in sponsoring a symposium held at Pasadena, California, on September 17-18, 1980. This symposium provided an opportunity for 300 people interested in storms and flood control systems to exchange views on the events of 1978 and 1980 and their effects on future flood hazard mitigation policies.

The program committee for the symposium consisted of:

Norman H. Brooks, Director, Environmental Quality Laboratory, California Institute of Technology (Director)
John F. Kennedy, Director, Iowa Institute of Hydraulic Research, University of Iowa, and member of the Committee on Natural Disasters, National Research Council
Jack Coe, Chief, South District, California Department of Water Resources
Daniel Davis, Section Head, Erosion Control Section, Los Angeles County Flood Control District
Robert Hall, Deputy Chief, Engineering Division, Los Angeles District, Corps of Engineers
Darwin Knockenmus, Subdistrict Chief, Water Resource Division, Laguna Niguel, U.S. Geological Survey
Robert C. Y. Koh, Research Associate in Environmental Engineering Science, EQL, Caltech
Brent D. Taylor, Senior Research Engineer, EQL, Caltech

Financial support was provided by:

National Science Foundation
(Grant No. PFR-7810631 to National Academy of Sciences, and subgrant No. C5588-80-365 from NAS to Caltech)

California Department of Water Resources
(Agreement No. B-53743 with Caltech)
U.S. Army Corps of Engineers, Los Angeles District
(Contract No. DACW09-08-M-2009 with Caltech)
U.S. Geological Survey (Geologic Division)
(Purchase Order No. 119573 to Caltech)
Frank and Elizabeth Gilloon Trust, and others
(Gifts to the Environmental Quality Laboratory
for discretionary use)

The California Department of Water Resources, the U.S. Army Corps of Engineers, the U.S. Geological Survey, and many other organizations (shown by the affiliations of the authors) also contributed significantly to the symposium through participation of their staff members.

The proceedings volume was edited and produced as a joint effort of the Environmental Quality Laboratory and the Committee on Natural Disasters. At EQL Theresa Fall coordinated the reviews and revisions of manuscripts and graphics; Brent D. Taylor, Vito A. Vanoni, and Robert C. Y. Koh assisted with the technical reviews. Norman H. Brooks organized the whole volume and prepared the summary paper. Debra Brownlie, Patricia Rankin, and Marcia Nelson handled the conference arrangements and secretarial work for EQL. At the NRC the final editing and preparation of the proceedings for publication was done by the Committee on Natural Disasters staff: O. Allen Israelsen, Executive Secretary; Steve Olson, Consultant Editor; Joann Curry and Lally Anne Anderson, Secretaries.

We gratefully acknowledge all of the various contributions of the authors, the staff, and the sponsors who made the symposium and the publication of this proceedings volume possible.

Symposium Co-Chairmen

Paul C. Jennings, Chairman, 1980
Committee on Natural Disasters

Norman H. Brooks, Director
Environmental Quality Laboratory

CONTENTS

OVERVIEW AND SUMMARY

by Norman H. Brooks

INTRODUCTION

Following the floods of 1978 and 1980 in southern California and Arizona a symposium was convened at the California Institute of Technology in September 1980 to document the significant events of these floods and to exchange information and evaluations. The symposium laid the groundwork for these proceedings, which serve as a compact permanent source of information on these floods for not only local readers but national readers as well.

Special attention is given to documenting problems--some engineering, some institutional--and to drawing conclusions and making recommendations for research. The papers included in this volume are not intended to be research papers or to replace the much more detailed reports of individual agencies. The emphasis was on preparing and presenting the papers soon after the event in such a way as to emphasize the regional nature of floods and flood control problems.

These proceedings are organized into several sections, with 35 papers altogether. Following this overview and summary, Section 2, STORM METEOROLOGY, which consists of four papers, describes the long-range weather patterns that affect the southwestern United States; the relationship of these patterns to sea surface temperatures in the North Pacific Ocean; the short-term synoptic meteorology of the storms under consideration, showing the importance of multiple storm sequences; and statistical analyses of return periods, based on historical data, for precipitation at a point.

Section 3, DOWNSTREAM RIVER FLOODING, consisting of nine papers, gives an overview of the floods on the larger rivers, how the flood control works responded, and what damages occurred. Section 4, UPLAND FLOODS AND SEDIMENT TRANSPORT (five papers), focuses on the unique aspects of sedimentation in regional floods. Section 5, LANDSLIDES, with four papers, explains the problems of landslides, both large and small, that were triggered by the prolonged periods of heavy rainfall.

Norman Brooks is Director of the Environmental Quality Laboratory and James Irvine Professor of Environmental and Civil Engineering at the California Institute of Technology in Pasadena, California.

Section 6, CASE STUDIES OF ENGINEERING PROBLEMS (four papers), gives detailed analyses of three particular engineering problems: the failure of levees on the San Jacinto River, the uncontrolled filling of Lake Elsinore to damaging stages, and the severe streambed scour threatening to undermine the Interstate 10 highway bridge over the Salt River at Phoenix, Arizona. The experiences and analyses described in these papers should be useful to engineers who deal with similar structures and situations in the future.

Section 7, EFFECTS ON THE SHORELINE, consisting of two papers, illustrates the damaging effects of the high storm waves and high tides that occurred in 1978 and 1980. Beach profiles shifted very rapidly, with sand being moved temporarily offshore, which exposed many shoreline structures to direct wave attack, causing severe damages.

Section 8, POLICIES FOR FLOOD CONTROL AND HAZARD MITIGATION (six papers), focuses on institutional issues. Four of these papers advocate a strong new emphasis on hazard mitigation, better flood warning systems, and other nonstructural approaches as part of the mix of society's activities to deal with floods.

About 300 people participated in the symposium, and many contributed to the questions and discussion. In the closing session there was a panel discussion by Russell Campbell, Engineering Geologist with the U.S. Geological Survey; John F. Kennedy, Director of the Iowa Institute on Hydraulic Research at the University of Iowa and member of the Committee on Natural Disasters of the National Research Council; Dale Peterson, Director of Community Services with the Federal Emergency Management Agency (FEMA) in San Francisco; and Richard Wainer, Los Angeles City Engineer's Office in Van Nuys. The writer served as moderator. Since it was not feasible to digest and record all of these discussions in this volume, I am attempting in this summary to capture the main conclusions and issues.* Nonetheless, the following conclusions and recommendations are solely the responsibility of the author and do not necessarily represent a consensus by the participants at the symposium.

For the record it should be noted that the following papers included in these proceedings were not presented at the symposium: "Geotechnical Origin and Repair of the Bluebird Canyon Landslide, Laguna Beach, California" by Beach Leighton and "Levee Failures and Distress, San Jacinto River Levee and Bautista Creek Channel, Riverside County, Santa Ana River Basin, California" by Joe Sciandrone, Ted Albrecht, Jr., Richard Davidson, Jacob Douma, Dave Hammer, Charles Hooppaw, and Al Robles, Jr. The latter paper is a shortened version of the official Corps of Engineers report on the San Jacinto River levee failure, which was not available in time for presentation at the conference.

Numerous brief discussions at the symposium are gratefully acknowledged, although very few are included in these proceedings.

---

*The entire symposium was recorded on 10 audio cassette tapes, which are available from the Environmental Quality Laboratory for the cost of duplicating.

HYDROLOGIC PERSPECTIVE

This section gives some general background on the flood hydrology of southern California and Arizona for those who may be unfamiliar with the area. An overview of the 1978 and 1980 floods is then presented in the next section, followed by discussion of nonstructural approaches and recommendations for research.

Flood Potential in the Southern California Coastal Region

Climate and Geology

The climate in the southwestern United States is arid, except for the California coastal strip and mountainous areas that receive orographic increases in precipitation. The main focus of this volume is the southern California coastal strip between Point Conception on the north and the Mexican border on the south, extending inland to the drainage divide between the streams flowing to the ocean and those flowing to the desert. The principal drainages are shown in Figure 1, and the identifications and areas are listed in Table 1. The elevation of the highest peak is about 3,500 m (11,500 ft) above sea level, and several are higher than 3,000 m (9,800 ft). The geology of the region, especially in relation to erosion and deposition, has been summarized by Fall (1981).

The mountain ranges are responsible for giving this strip a semiarid Mediterranean climate with considerably higher rainfall (an annual average of 10 to 25 in. or 250 to 630 mm in the valley areas and up to twice as much in the mountains) than on the desert side of the mountains (with less than 8 in. or 200 mm). The mean annual rainfall distribution for California is shown in Figure 2. The large variation of the annual rainfall at Los Angeles for the period 1877-1980 is shown in Figure 3 of the paper by James Slosson and James Krohn in Section 5. The precipitation falls almost entirely during the winter months, with long dry hot summers that generally inhibit the development of forest cover below about 1,500 m elevation except on some north-facing slopes. Below this level the slopes are covered with chaparral (native brush), a few trees, grasses, or bare soil. The soils in the mountains are quite thin and rapidly erode or slide down the slopes; the underlying rocks decompose fairly rapidly, yielding an overall long-term erosion rate of the order of 1 m per thousand years (Taylor, 1981). The vegetation and soils of the area are described in more detail by Wells and Palmer (1981). A comprehensive summary of a wide range of hydrologic and geologic characteristics for a part of the San Bernardino Mountains has been lucidly presented with excellent maps and graphics by Troxell et al. (1954).

Flood Factors

Several factors make this region susceptible to severe floods and storm damage:

1. Steep slopes in the mountains, with many slopes at the angle of repose (or steeper) for loose material. Landslides and mudflows are common during heavy and prolonged rainfall, and landslides may occur up to a year later.

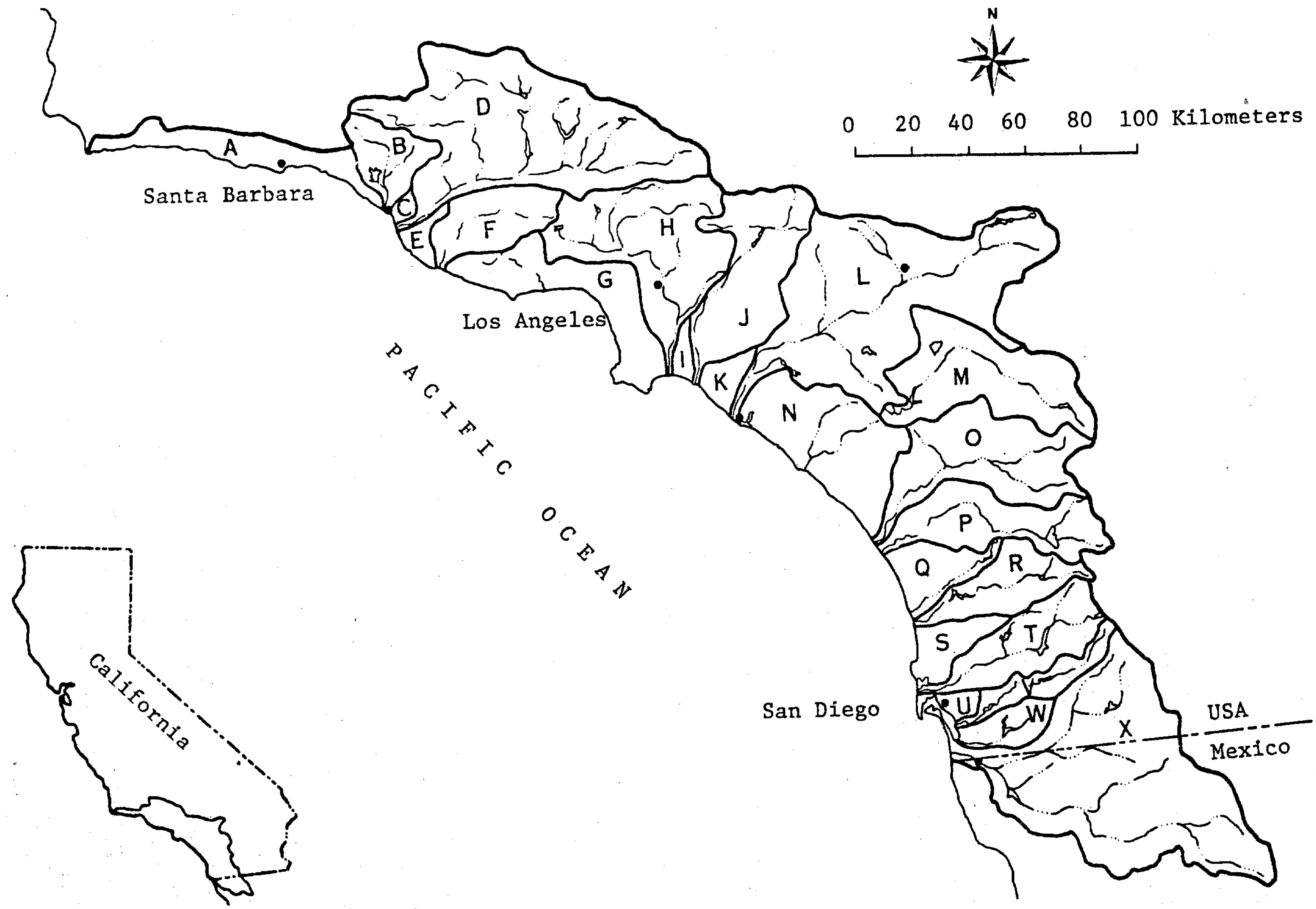


FIGURE 1 Southern California coastal area showing the principal drainage units, identified by letter code and listed in Table 1. Source: Taylor (1981).

TABLE 1 Major Drainage Units in the Southern California Coastal Area (as shown in Figure 1)

Map Symbol	Principal Basin or Group of Small Basins	Controlled Drainage Area of Principal Basins[a] (sq km)	Area (sq km)	Percent of Area Controlled in Principal Basins
A	Santa Ynez Mountains group	--	901	--
B	Ventura River basin	243	585	42
C	Ventura group	--	52	--
D	Santa Clara River basin	1,527	4,219	37
E	Oxnard group	--	159	--
F	Calleguas Creek basin	--	837	--
G	Santa Monica Mountains group	166	1,493	11
H	Los Angeles River basin	866[b]	2,155	40
I	Long Beach group	--	120	--
J	San Gabriel River basin	1,400	1,663	84
K	Huntington Beach group	--	234	--
L	Santa Ana River basin	3,950	4,406[c]	90
M	Lake Elsinore basin	1,989	1,989[d]	100
N	Laguna Hills group	--	1,737	--
O	Santa Margarita River basin	958	1,927	50
P	San Luis Rey River basin	531	1,450	37
Q	Escondido Creek group	--	568	--
R	San Dieguito River basin	785	896	88
S	San Clemente Canyon group	--	437	--
T	San Diego River basin	686	1,119	61
U	San Diego group	--	157	--
V	Sweetwater River basin	471	567	83
W	Otay River basin	255	370	69
X	Tijuana River basin	3,175	4,483	72
Totals		17,002	32,524	53

[a]Calculated by adding the drainage areas controlled by the major water retention structures that are farthest downstream in each basin.

[b]Whittier Narrows flood control basin controls both Los Angeles and San Gabriel rivers. This estimate assumes that 35 sq km of the drainage area controlled by the Whittier Narrows structure lies within the Los Angeles River drainage basin.

[c]Excludes Lake Elsinore basin (M).

[d]Closed interior basin. Overflow into Santa Ana River basin did not occur between 1916 and 1980.

Source: Brownlie and Taylor (1981).

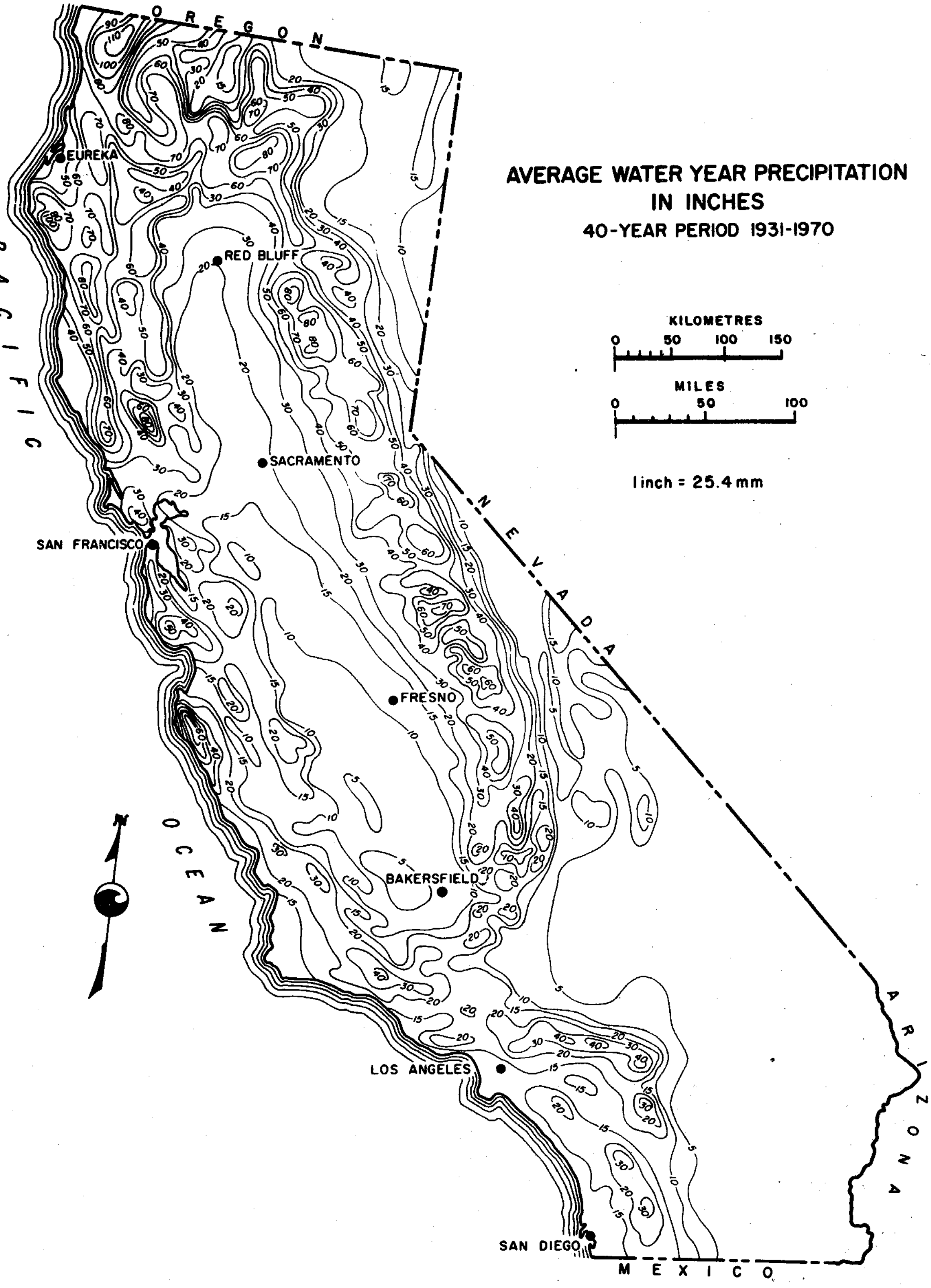


FIGURE 2 Annual precipitation map for California. Source: California Department of Water Resources (1980), p. 16.

2. Intense winter storms, often in groups (such as six in nine days in February 1980), with strong orographic increases in precipitation with elevation.

3. Snowfall generally above 2,000 to 2,500 m, an area that is a minor fraction of the total area. Floods are therefore caused by rapid rain runoff, not by snowmelt.

4. Naturally high erosion rates (or sediment yields), causing very high sediment transport out of the canyons onto alluvial fans and floodplains.

5. Burned watersheds, producing flood peaks that are several times higher and sediment outflows that are an order of magnitude greater than for unburned watersheds.

The fire-flood sequence is the most devastating and least well controlled of the flood phenomena of southern California and contributed significantly to the damages to the foothill areas in the 1978 and 1980 storms (see the papers by Wade Wells and Daniel Davis in Section 4). The chaparral on the lower slopes burns fiercely when fires start accidentally in the dry weather of late summer or early fall, often whipped by Santa Ana winds from the north off the desert. Many residents of southern California living next to the foothills have luckily escaped the damage of the summer fires only to see their property buried by sediments pouring out of the canyons or sliding down slopes in the winter floods.

The inhabitable land on the coastal strip naturally lies between the mountains and the shoreline. Before human development these lands were largely depositional areas; although the alluvial fans at the mouths of many canyons were the most rapidly aggrading features, many have nonetheless become urbanized areas (such as Altadena, shown in Figure 3). The fans may have slopes of up to 0.08 to 0.1. The main rivers in the valleys still are relatively steep, with slopes of 0.001 to 0.01--large values for major rivers that make them flow at relatively high velocities, often with wavy surfaces. Before human intervention the gravel and coarse sands were all deposited on the alluvial fans and the river valleys, while much of the fine sand, silt, and clay was carried through to the ocean in large uncontrolled floodflows. This flow of sand has been the principal source of nourishment for southern California's extensive beaches (Brownlie and Taylor, 1981).

Flood Control

The early settlers in the coastal areas of southern California quickly discovered how brutal uncontrolled streams and rivers could be. The earliest flood control efforts were accelerated by the formation of the Los Angeles County Flood Control District in 1915, which had as its mission not only flood control but also water conservation. Since that time the district, along with the Corps of Engineers (starting in the 1930s), has built one of the most intensive systems of flood control structures in the world. Outside Los Angeles County the flood control systems are less developed, with more works in the planning stages to protect growing developments.

In the early years the flood control systems in the Los Angeles area focused on major flood control dams and channel improvements, most with

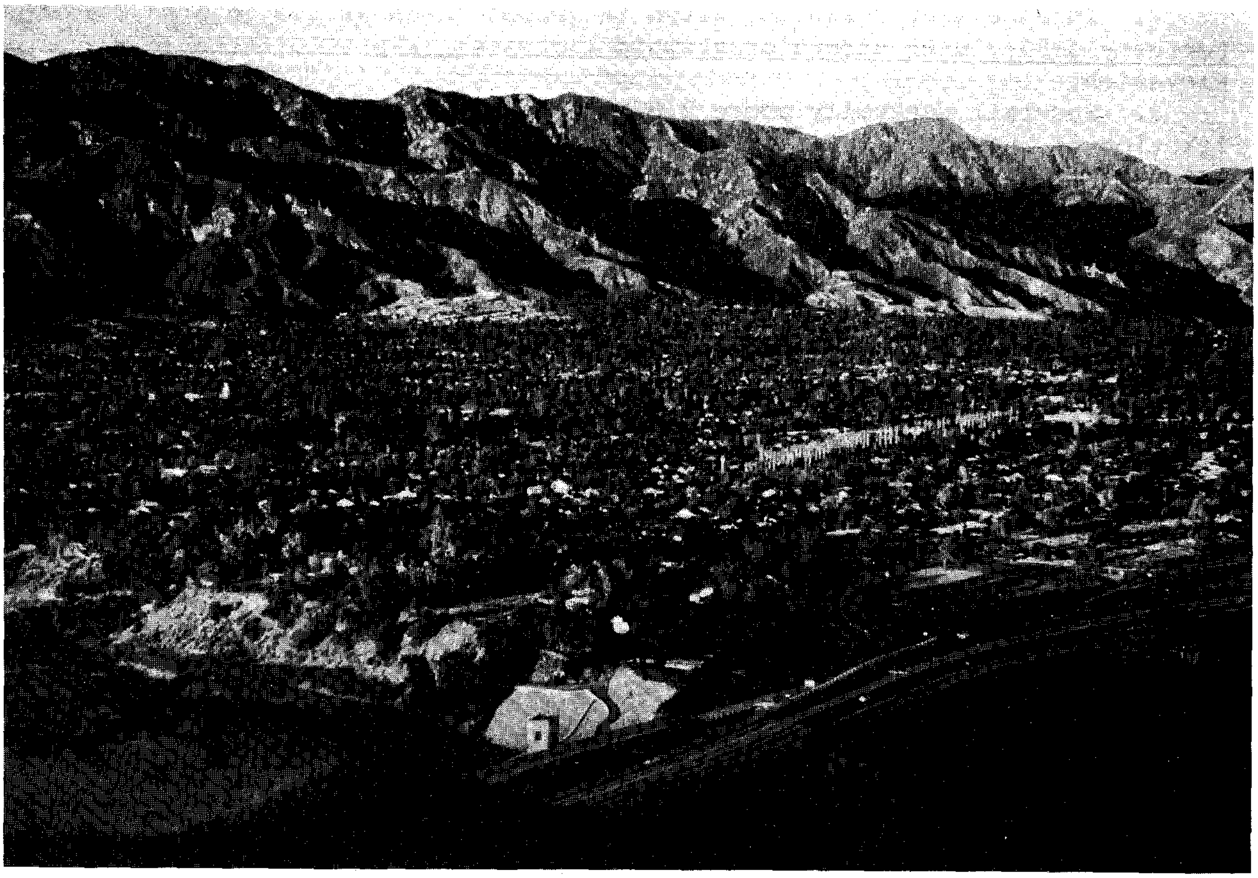


FIGURE 3 The San Gabriel Mountains drain from steep canyons directly onto alluvial fans, such as this large one underlying Altadena and the northern part of Pasadena (northeast of Los Angeles). The developed areas on this fan are protected by debris basins at the mouths of the canyons (see Figure 4).

permanent concrete linings. However, after the New Year's Day flood in 1934 it became apparent that extraordinary measures would be needed to control the huge and damaging outpourings of sediment (or debris) from the many smaller canyons onto the urban areas in the foothills. A system of 105 debris basins was conceived, and most of them have now been built. The longest period of operation is now over 40 years, so some statistics on rates of filling are becoming established (see the paper by Daniel Davis in Section 4 and Brown and Taylor (1981)).

A typical basin is shown in Figure 4, and design details are shown in Figure 1 of the paper by John Tettemer in Section 4. As sediments accumulate these basins are supposed to be excavated, sometimes even between storms (see the paper by Daniel Davis in Section 4). They are intended only to catch the coarser sediments, with the finer sediments flowing through the outlet tower (see Figure 1 in the paper by John Tettemer). They have insignificant water

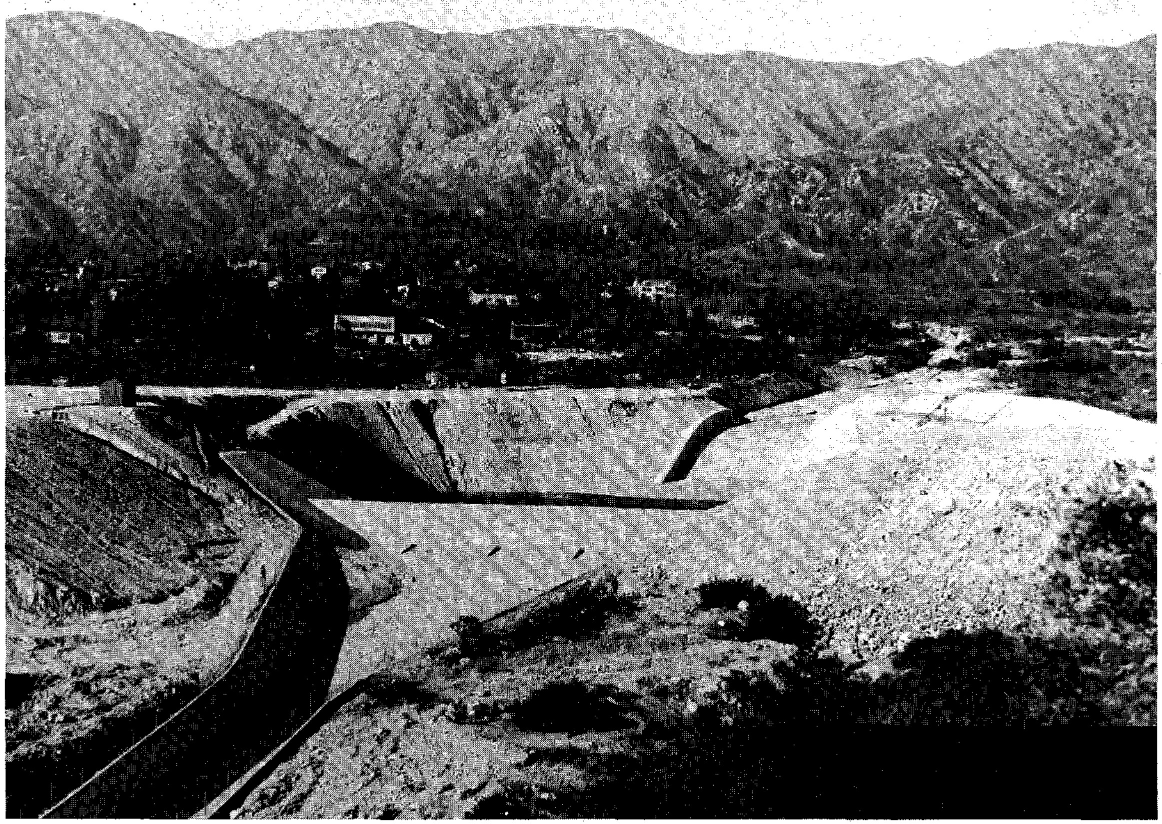


FIGURE 4 Pickens debris basin in La Crescenta, California, shortly after it was constructed by the Los Angeles County Flood Control District in 1936. Flow enters from upper right, and after coarse sediments are captured the outflow passes into a lined channel (lower left).

storage volumes and do not appreciably change the water discharges. These flows can then be carried in lined concrete channels without danger of the channels being filled by debris. Earlier efforts to convey canyon floodflows across alluvial fans without removing the debris met with quick and unequivocal failure--channels simply filled right up with sediments, allowing the water to flow over the fan as before (see the photograph in Figure 5, taken after the 1938 flood).

Large flood storage dams have also been filling up at a rapid rate, and many of them have had to be cleaned out about once every 30 to 50 years. Disposing of all of the sediments from the major dams and the debris dams is posing an increasing problem for the Los Angeles County Flood Control District and other agencies because there are few available places to store this material safely for the long run. The historical data on cleanouts of major reservoirs, debris basins, and channels (before the 1978 and 1980 floods) have been summarized for the southern California coastal region by Kolker (1981).

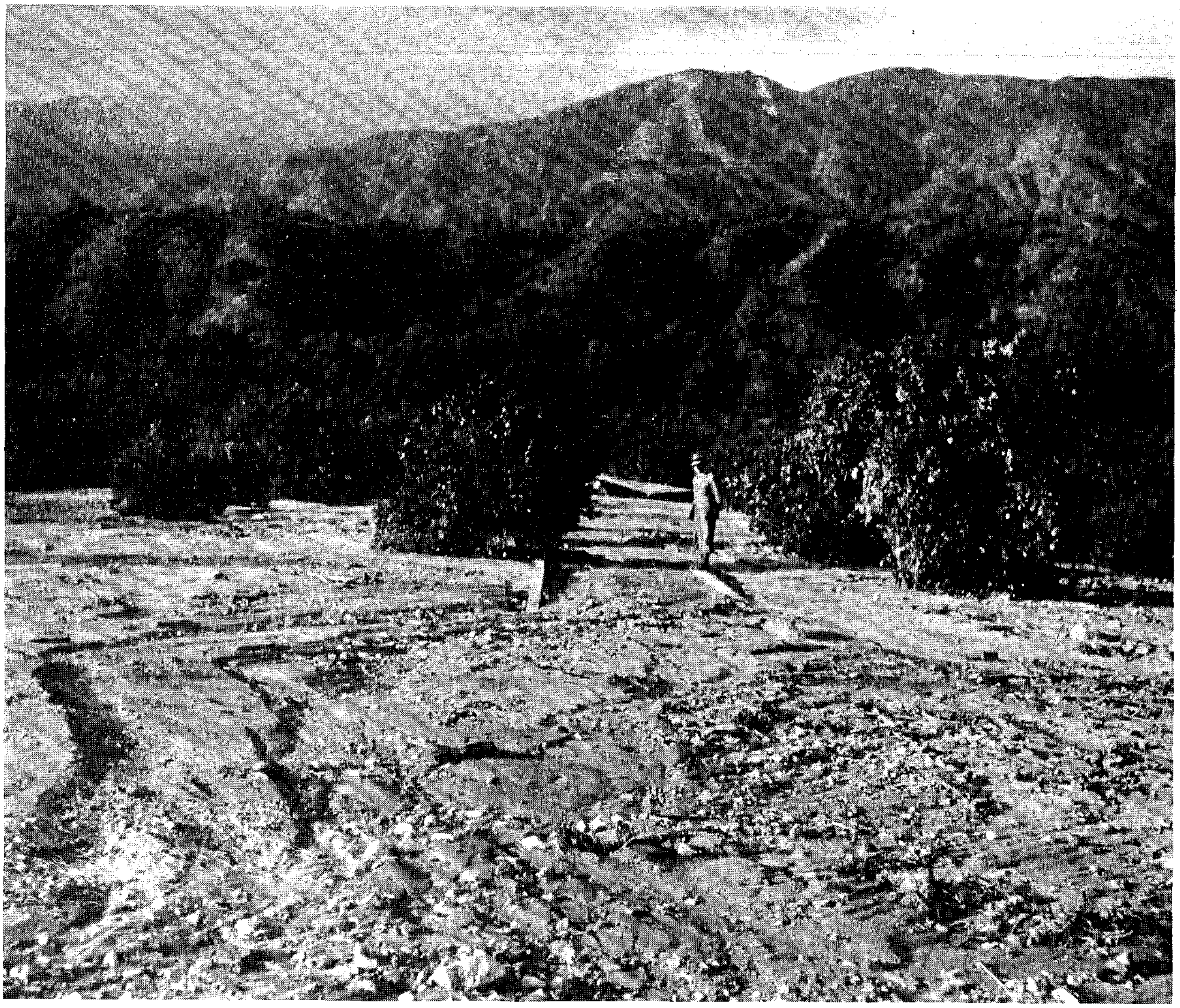


FIGURE 5 Concrete flood channel on an alluvial fan in Monrovia, completely filled with sediment in the 1938 flood (only a short length of the very tops of the channel walls is visible). Without an upstream debris basin a channel like this is useless in a flood.

Flood Potential in Arizona

In Arizona the rainfall from winter storms from the Pacific Ocean is generally less than in coastal portions of California. On the other hand, short-duration intense rainfall from thunderstorms is more frequent. Occasionally, Arizona is also hit with intense rain from tropical storms that come from the south off the Gulf of California and Pacific Ocean during the fall. The primary area of interest in this volume is the vicinity of Phoenix and the upstream tributary areas of the Gila River system as shown in Figure 1 of the paper by B. N. Aldridge, Section 3. In these areas, as well as in California, erosion and sediment transport increase the flood hazards.

As urbanization spreads around Phoenix and other areas in the arid Southwest (e.g., Palm Springs, California, or Tucson, Arizona), developers will be looking for choice building sites and will think that many alluvial fans are attractive for development. In his two papers in this volume, John Tettemer describes the urgency of adopting a flood mitigation policy for floodplain zoning in order to keep developments off those alluvial fans that are active, hazardous, and entail exorbitant costs of protection. The development of floodplain hazard maps along with the implementation of the National Flood Insurance Program by FEMA will be very useful in forcing communities to pay more attention to sediment hazards.

OVERVIEW OF THE 1978 AND 1980 FLOODS

The notable flood events of 1978 and 1980 are discussed in the papers that follow. Our job here is to ask "What did we learn?" and "How can we improve our systems for flood control and damage mitigation?" This subject will be discussed in the next several sections; since this is an overview and evaluation, the reader is referred to the papers for detailed information. A discussion of nonstructural approaches and recommendations for research will be presented in later sections.

The Natural Events--How Well Do We Understand Them?

The storms and floods of 1978 and 1980 have each been judged to be of the size that can be expected approximately once in 25 years (although the severity of these events varied considerably with location). Precise frequencies cannot be determined because our data base is too short and different stations and criteria give different answers. Whether the number is 10, 25, or 50 years, these floods were well within the range of frequencies for which the flood control systems have been designed. They were definitely not of disastrous proportions (say, once-in-several-thousand-years frequency) that would exceed the capacity of the control structures. Therefore, without minimizing the loss of life, property damage, and general disruption and psychological impacts that did occur, it is important to realize that these storms were far from the worst that could occur.

In 1978 the two major storms occurred separately (in February and early March) on watersheds well saturated with previous rainfall. In 1980 the biggest floods were caused primarily by an unusual sequence of six storms in the eight and a half day period February 13-21. Figure 6 shows the hourly distribution of the 19.71 in. (501 mm) of rainfall that fell in that period at Caltech, while Figure 7 shows the accumulative amounts. For short-term durations the amounts were generally far from record-breaking (see the paper by Wade Wells in Section 4), thereby indicating that the main flooding problems in 1980 were not associated with small drainages or culverts but rather with the larger-scale flood control dams and channels of the bigger systems. The exceptions were those watersheds that had been burned within a few years prior to 1980 (see the paper by Daniel Davis in Section 4).

The meteorology of these situations is now much better understood than it was before, both on short and long time scales. Satellite observations help greatly in understanding the sequences of storms (such as occurred in 1980)

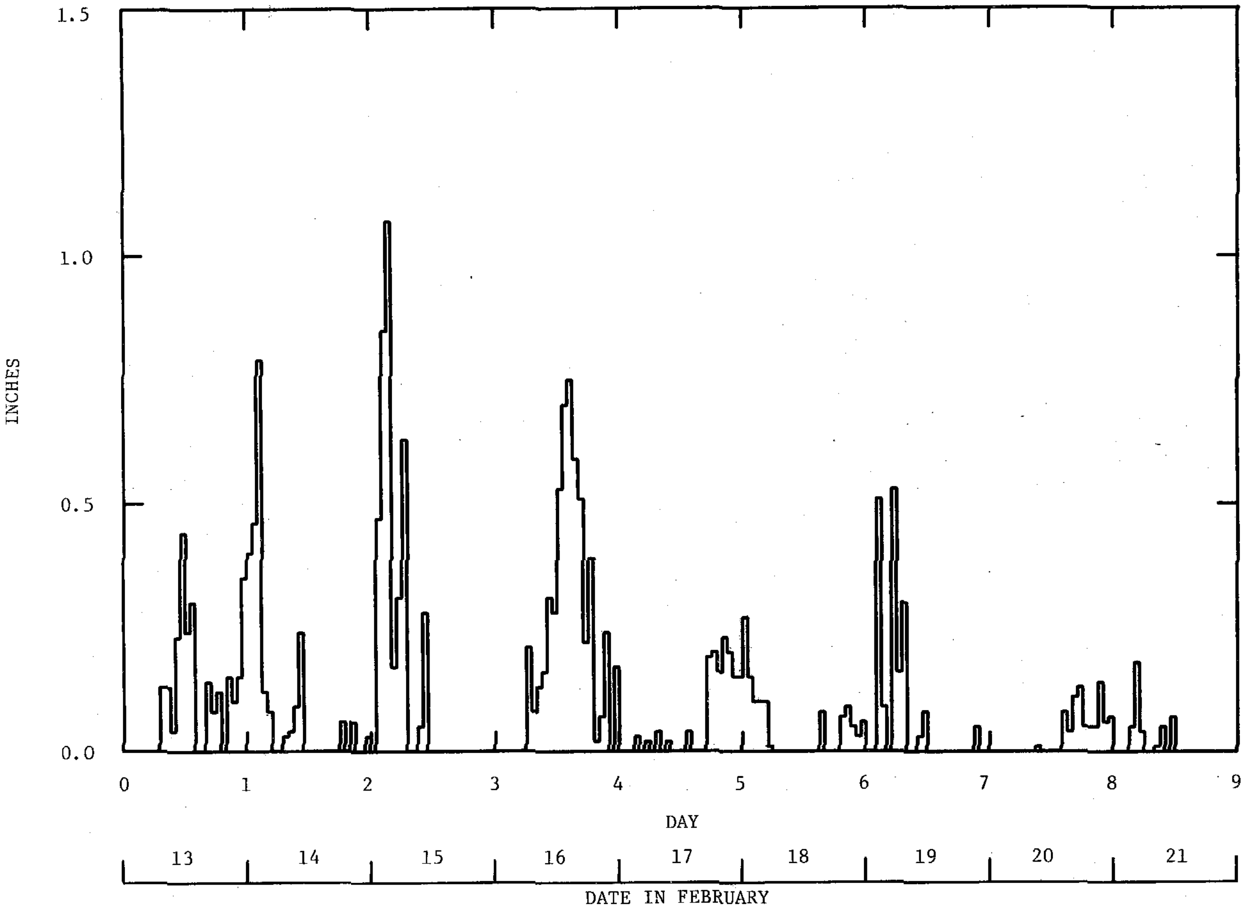


FIGURE 6 Hourly rainfall at the California Institute of Technology in Pasadena for the six storms in the period February 13-21, 1980 (from the recording gage record of Station 303F operated by Caltech for the Los Angeles County Flood Control District).

and in predicting their arrival times and approximate intensities. The regular "clear water" hydraulics of stream runoff is well in hand, except for the sharper and higher peaks coming from urbanized areas as more surfaces get paved or roofed (see the paper by Philip Pryde in Section 3).

Heavy sediment transport in floods from the canyons is always expected and is part of the long-term geologic process that downcuts the mountains at a rate of about 1 m per thousand years (while tectonic processes uplift them at a rate several times higher). In fact, much of the development in southern California lies on active or historical depositional areas. In the extensive recently burned areas the sediment erosion rates were increased as much as tenfold over unburned areas (see the paper by Daniel Davis in Section 4); floodflows were also sharply increased due to bulking (high sediment loads), less infiltration, and faster flows (Wells, 1981). Practically all the flood damage in the foothill areas in 1978 and 1980 was associated with burned watersheds.

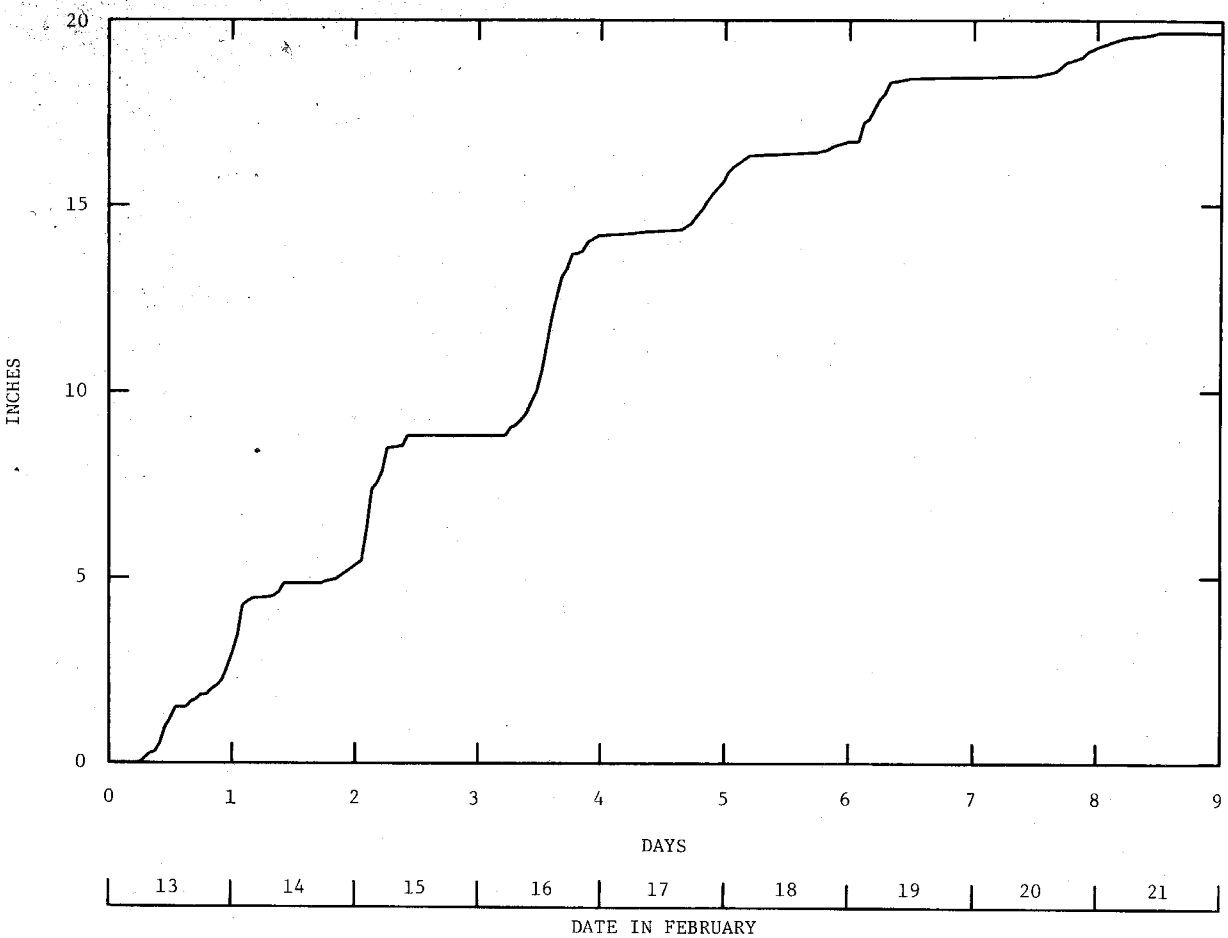


FIGURE 7 Cumulative rainfall at the California Institute of Technology for February 13-21, 1980.

Fires, which are natural for the southern California mountains, occurred before humans developed the area and recur in spite of our efforts at suppression (see Wells (1981) for a discussion of fires). Available fuel in the chaparral stand builds up between fires so that after several decades without a burn, it is practically impossible to stop a wildfire before it covers tens of square kilometers. In the long-term geologic sense the heavy erosion following fires (which also occurred before man's arrival) may be considered part of the normal process of downcutting. The fire-flood sequence will continue to be a threat to foothill communities, and the risks of these events are probably underestimated by the public.

Although the most spectacular sediment transport by streams is in the mountain canyons, even the downstream rivers can produce staggering rates of transport of suspended sediment. For example, data in Kenneth Wahl's paper in Section 3 for the Santa Clara River, the region's largest, show instantaneous sediment transport rates of over one million tons per day and sediment concentrations ranging up to 32 grams per liter.

Landslides and mudslides are predictable general consequences of wet winters in southern California. Small landslides occur as soon as the ground is saturated, while larger slides do not occur until months later because of the time required for the deep percolation of moisture to the weak shear zones. One example is the Bluebird Canyón landslide in Laguna Beach, which destroyed 25 homes on October 2, 1978, seven months after the end of the rainy season (see the paper by Beach Leighton in Section 5). An even longer delayed response was the large Malibu rock slide that occurred on April 13, 1979, blocking Pacific Coast Highway more than a year after the heavy rains (see the paper by Raymond Forsyth and Marvin McCauley in Section 5). We can identify areas that are prone to landslides and mudflows, but we do not have the ability to predict just when any particular slide might occur. Strict controls on hillside developments, such as the ordinances adopted by the City of Los Angeles (see the paper by James Slosson and James Krohn, Section 5), can significantly mitigate the hazards from these natural phenomena.

The shoreline in southern California, especially in the vicinity of Malibu, received heavy wave attacks during the 1978 and 1980 storms. George Armstrong in his paper in Section 7 describes the shore erosion of 1978 as the worst in the past 40 years, but he still calls the 1978 storm season "exceptional but not unusual." As a predictable natural process during winter storms, the large waves caused a major realignment of beach profiles, shifting sand from the beach to the offshore berm and leaving many structures unduly exposed to the breaking of waves. The seasonal coming and going of beaches is a normal phenomenon, as described in the paper by Martha Shaw in Section 7. She observed that during the February 1980 storms over 150 cubic meters of sand per meter of beach were removed in a few days from the nearshore region of Leadbetter Beach at Santa Barbara; this is equivalent to the removal of an area of 150 sq m in the vertical cross section. Again, these are normal well-understood phenomena, but the risks due to shifting beach profiles during storms are probably generally underestimated.

Flood Control Structures--How Well Did They Work?

In California the floods caused 38 deaths in 1978 and 18 deaths in 1980; estimated property damages were $220 million in 1978 and $270 million in 1980 (see the paper by Carlos Garza and Craig Peterson in Section 2 and Jacob Angel's paper in Section 8). However, Joseph Evelyn in his paper in Section 3 estimates that the Corps of Engineers projects alone in the Los Angeles-San Gabriel-Santa Ana River systems in southern California prevented more than $4 billion in damages. In Arizona the flood damages were $70 million in March 1978 and $90 million in December 1978; no estimates were given for 1980 (see the paper by B. N. Aldridge in Section 3).

In general, the main flood control systems in southern California and Arizona performed very well. Yet there were some failures and problems with engineered systems, in spite of the highly favorable operating experience.

Levee Failures

Levee failures on the San Jacinto River flooded the town of San Jacinto; other failures on Calleguas Creek flooded the Point Mugu Naval Air Station.

At San Jacinto the levee failed due to toe erosion, while at Calleguas Creek the levee was overtopped.

The levee failures on the San Jacinto River are fully described in the papers by Kenneth Edwards and Joe Sciandrone et al. in Section 6. The apparent cause of the failure was undermining of the levee toe due to very deep scour. The location of the scour was associated with the confluence of Bautista Creek and the San Jacinto River, which caused a poor alignment of the main stream of flow with respect to the levee. The peak flow in the channel (25,000 cu ft/s) was only 29 percent of the design flow (86,000 cu ft/s). The median size of the riprap rock that was specified at the time of construction of the levee was 130 lb (12 in.), whereas present Corps of Engineers criteria would have called for 2,000-lb (30-in.) rock (for details see the paper by Joe Sciandrone et al. in Section 6).

These examples illustrate that channels having sand beds with levees may not be as safe as the designers expected. Even grade control structures, such as those in the Santa Ana River in Orange County, may not control degradation in cases where the stream is starved for sediment (see the paper by Carl Nelson in Section 3). The failure of such drop structures can be followed by undermining of levees.

Bridge Piers Undermined by Channel Scour

The undermining of bridge piers is another recurring engineering problem, as illustrated by several failures in San Diego County, the problems with the Interstate 10 bridge at Phoenix, and near failures on the Santa Ana River in Orange County. During floods, scour may reach considerable depths, often much more than the depth of the water itself. The depth of scour is dependent on the amount of sediment load of sand and gravel sizes entering the channel with the water discharge. Channels with sand beds downstream of storage dams (e.g., the Santa Ana River below Prado Dam; see the paper by Carl Nelson in Section 3) are especially vulnerable to severe degradation because almost all of the sand load is probably deposited in the reservoir. The discharge of water without a sand load attacks the bed as it seeks to establish a new equilibrium rate of transport. Urbanization may also lower the input of sand in valley and hill areas below previous natural rates.

Sediment-control structures like debris basins, which are absolutely essential for preventing severe aggradation on alluvial fans, may create a hazard of severe degradation unless they feed into lined channels or unless the channels have other sources of sediment to keep them in reasonable balance.

Increased Flood Peaks from Urban Areas

Spreading urbanization is tending to reduce the concentration time (or the time from peak rainfall to peak streamflow) and to increase the peak flood discharge for a given storm (see the paper by Dolores Taylor in Section 3, which indicates that this factor contributed to the overtopping of the Calleguas Creek levee). This effect is reducing the protection of the existing set of improved channels, inasmuch as they will not be able to carry floods of lesser frequency than originally thought (see Philip Pryde's paper in Section 3).

Overflow of Debris Basins in Fire Areas

In Los Angeles County the severe floods and debris transport from burned areas exceeded the capacity of some debris control structures (Daniel Davis gives examples in his paper in Section 4). The present design criterion of 200,000 cubic yards of capacity per square mile (or 59,000 cu m/sq km, which is equivalent to 5.9 cm of depth over the watershed area) appears to be adequate for the storms that occurred, according to Davis, who shows no measured values exceeding 50,000 cu m/sq km. However, some of the debris basins were built with smaller volumes in earlier years and can be expected to overflow more often (e.g., upper Shields Canyon). For watersheds that were not recently burned, the debris basins in the Los Angeles County system proved to be very sufficient, with no problems during the 1978 and 1980 floods.

Flooding and Sediment Damages in Unprotected Areas

Streams and Canyons

Other flood problems occurred in flood-prone areas unprotected by flood control structures, such as areas upstream of debris basins and dams or houses built in canyons in the Santa Monica Mountains and elsewhere. There the pattern of development of many houses along the canyon bottoms makes flood control impossible. When these streams are aggraded during floods because of heavy sediment loads (later the deposits will be cut out again), flooding of roadways and dwellings is almost inevitable. Here the problem is not with the flood control system but rather with a lack of control of development in areas of extreme flood hazard.

Lake Elsinore--Flooding of Developments Encroaching on the Historical Lake Area

A unique flood event in southern California was the record high level that Lake Elsinore reached in March 1980, which caused extensive flooding and threatened the developments that had gradually encroached on the historical lake area (see the paper by Charles White in Section 6). Lake Elsinore is the sink for the San Jacinto River and has a relatively high overflow channel to the Santa Ana River system. In geological time this channel undoubtedly carried overflows a number of times. However, it had been so long since Lake Elsinore had filled up (not since 1916) that the perception of a flood hazard had all but faded away! Damage prevention would have been easy with proper zoning control of the developments around the lake. Present zoning controls, stringently enforced, will reduce flood damages in the future.

Landslides and Mudflows

Landslides

Landslides were widely scattered during and after the storms, threatening loss of life as well as property. There is no practical way to stop a landslide once it starts, so all countermeasures must be preventive. During a storm, individual troublesome slopes can be protected from additional rainfall by plastic sheeting or by deflecting concentrated surface runoff away from weak slopes, if possible. But only vigorous zoning and grading ordinances, such as in the City of Los Angeles, can permanently reduce the potential for landslide damage. Hazards can come either from natural slopes or from improperly constructed earth embankments. Structures at both the tops and bottoms of the slopes are in jeopardy.

James Slosson and James Krohn report in their paper in Section 5 that the City of Los Angeles has been keeping detailed statistics of landslide damages within the city and relating these to the ordinances in effect at the time of development. Total damages within the city were estimated to be $50 million in 1978 and $70 million in 1980. Their Table 3 (showing 3,000 failures for 1978) gives a slope failure rate of 7.5 percent for pre-1963 construction (before the modern code) versus only 0.7 percent for post-1963 construction. Damages in 1978 to developments under the new code are estimated to have been reduced 95 percent from what they would have been had the new code not been adopted in 1963.

The essence of the code is to require proper geologic investigations of natural slopes and avoid building where there are significant hazards. For man-made embankments it requires proper soil mechanics engineering regarding materials to be used, choice of slopes, and methods of construction. Furthermore, geologists and soil mechanics engineers must inspect grading projects while they are in progress and certify them upon completion as meeting the safety standards.

While the present codes effectively prevent construction of new possible sources of damage, houses built before 1963 could still be subject to heavy damage in future wet years under the right circumstances. According to Harold Weber, Jr., in his paper in Section 5, shallow slides may be triggered by special sequencing of rainfalls. For instance, over 100 homes were damaged in Monterey Park on February 16, 1980 (the day of heaviest rain--see Figure 6 above), although there had been no previous damage in over 40 years since development of the area started. In other areas damage regularly occurs in any very wet year, and for some areas damage was much worse in 1978 than in 1980.

Mudflows

When a saturated landslide begins to liquefy and flow like a viscous fluid, it is called a mudflow. In the mountains, landslides often fall into streams in the canyon bottoms and may start mudflows, which surge down the natural stream channels. These mudflows have the consistency of wet sloppy concrete, with large boulders and gravel included in the matrix. They stop as soon as they spread out laterally or the grade flattens, and the water and fine sediments drain away from larger sediments as they stop.

Mudflows at the base of hillslopes can flow out with flatter surface slopes than landslides per se. Since the National Flood Insurance Program covers <u>mudflow</u> damage but not <u>landslide</u> damage, there is a difficult problem of definitions. Physically, however, a sharp distinction is often not possible--who can say exactly where a landslide turns into a mudflow? Mudflows may also start as a surface or streambed erosion process on very steep slopes during periods of exceptionally heavy rainfall without being triggered by a landslide. A committee of the National Reseach Council has prepared a report for the Federal Emergency Management Agency on methodologies to define and clarify mudflow hazards and distinguish them from landslides for insurance purposes (National Research Council, 1982). For an excellent description and explanation of landslides and mudflows in the Santa Monica Mountains, see Campbell (1975).

Mudflows should not be confused with heavy sediment transport and deposition by streams during floods. Mudflows are special, distinct episodes and are not continuous like floodflows. The alluvial fans at the mouths of mountain canyons are mainly the result of stream transport and deposition, not of mudflows. Although sediment concentrations in mudflows may be over 1,000 grams per liter, much more sediment transport occurs in alluvial floods (with sediment concentrations only very rarely exceeding 100 grams per liter) because of the latter's high volume. Again, there may be instances where the distinction is unclear.

NONSTRUCTURAL APPROACHES TO DAMAGE REDUCTION--WERE THEY USEFUL IN THE 1980 FLOODS?

Risks and Benefits

There is a growing awareness that flood control structures (dams, lined channels, storm drains, pump stations, etc.) are necessary but not sufficient to provide for safety and prevent damage (e.g., see California Department of Water Resources (1980) and the paper by Ronald Robie in Section 8). Nonstructural approaches, which are getting increased attention, will be discussed in this section. There are several compelling reasons for this shift in attitudes toward flood control:

1. <u>Flood control structures can be designed to handle floods only up to a certain size</u>, usually expressed as a flood frequency. For floods exceeding this size the structures may no longer be effective or, in case of failures, the damages can be worse than if there had been no structures at all. For example, a levee designed for a 25-year flood may create confidence that encourages development next to the levee; then if a 50-year flood causes the levee to fail the damage might be extensive. Although spillways of major dams may be designed for very large floods (the maximum possible as determined by hydrometeorological methods), the channels downstream often cannot feasibly be built to carry such extraordinary floods. Acceptance of some risk is inevitable and economically sensible. At some point on the scale of risk reduction, flood insurance and disaster assistance provide a way to share the remaining risk at annual costs to society that are less than the costs of additional structural measures.

2. <u>The cost of public works has increased sharply in the last decade</u>. Not only has the cost of construction increased by a factor of about three

over the last decade, but the cost of borrowing (expressed as the interest rate paid by government) has also tripled. Therefore the annual cost could have increased between three and nine times, depending on the length of the repayment period. Thus there are strong economic incentives to consider and use other approaches.

3. The environmental impacts of flood control works are being viewed with more sensitivity than they were 15 to 20 years ago.

4. Experience, including that with the floods of 1978 and 1980, is showing that nonstructural methods can be used effectively to save lives and reduce property damage for reasonable costs.

5. Some nonstructural measures, such as better flood forecasting and better flood channel maintenance, enhance the protection afforded by structures already built.

In this section we shall discuss some nonstructural measures of flood control, both as they were used in 1980 and as they might be used effectively as a more significant part of an overall response to floods in the future.

The 1980 Experience with Nonstructural Approaches

Flood Predictions and Warnings

With satellite imagery the National Weather Service was able to make better storm predictions in 1980 than ever before. However, since the intensity of rainfall and small-scale variations are still difficult to predict, it is useful to instrument the key larger watersheds with real-time telemetry to transmit rainfall amounts and stream stages from upstream locations to a central operations center. Using computer simulation, downstream hydrographs can be predicted in time to warn residents and mobilize flood fighting forces. In their paper in Section 3, Ira Bartfeld and Dolores Taylor describe the development of such a system for the unregulated Sespe Creek in Ventura County after the 1978 floods. In 1980 the system was operational and was instrumental in saving Fillmore from a repeat of the damaging flood and the frantic evacuation it experienced in 1978.

Operation of Flood Control Systems

Although all major reservoirs performed well and prevented millions of dollars in damage (see the paper in Section 3 by Joseph Evelyn), there is still need for a more systematic approach to reservoir operations to get the most benefit from the overall system of reservoirs and channels. Although the storms of 1978 and 1980 were not a truly great series of storms, the larger flood control dams and channels were used in many cases to near capacity in 1980. In a system of storms with a return period of approximately 100 or more years, the writer believes that there would be some significant uncontrolled spillway releases, with some downstream channels likely to overflow since they generally have less capacity than do the spillways of large dams.

With telemetry of flood data to a computer during a flood, the best strategies for releases on multidam systems could be calculated while considering the limitations of the downstream channels.

Flood Fighting

Flood damages can be reduced by carefully patrolling flood channels, levees, debris dams, and other flood control works. In case of trouble, fast responses can often be vital--for example, in removing trash that plugs an outlet or channel. In Santa Barbara County a diligent patrol of levees on the Santa Maria River probably averted a levee failure when deteriorating sections were discovered and emergency reinforcement procedures were instigated immediately (see James Stubchaer's paper in Section 4).

During the floods of 1978 and 1980 local officials received a great many calls for assistance from private property owners with problems of high water, deposition of debris, or erosion. Personnel of flood control agencies and public works organizations generally do not have the authority (or the time during floods) to provide emergency flood protection on private property, a fact that is not generally understood by the public. Since the City of Los Angeles had no way to respond to the numerous requests for help, callers were referred to the TreePeople, a private volunteer organization primarily dedicated to planting trees and other conservation projects (see the paper by Andrew Lipkis, Sherna Hough, and Lisa Geller in Section 8). In a very short time (without any advance planning) the TreePeople established a telephone hotline and mobilized hundreds of volunteers to help people protect their houses and property with sandbags and other small-scale emergency measures. The volunteer organization's response was so successful that it should serve as an example for flood fighting during the next flood and in other areas. Some advance organizational work and training of team leaders would be very useful to make the volunteer work as effective and safe as possible.

Temporary Defensive Measures in Fire Areas

When a watershed burns, the flood and sediment hazards are greatly increased. Flood control agencies can make special efforts to warn property owners of the extra hazards and advise them of temporary precautionary measures to take until vegetation reestablishes itself on the watershed over several years. Temporary public works can be erected to retain sediment, and flood fighting preparations and evacuation plans can be made. A program of this kind was successfully implemented following the Sycamore Canyon fire near Santa Barbara in 1977 (see James Stubchaer's paper in Section 4).

Cleanup and Maintenance

Agencies have learned that good maintenance of flood control facilities between floods is essential to keep the floodflow capacities of the structures up to design values. Such maintenance includes removal of sediment and debris from debris basins, reservoirs, and flood channels; repair of levees and other structures; and upkeep of outlet works and pump stations. Local agencies have the responsibility for maintaining flood channel projects built by the Corps of Engineers, but they may not have sufficient funds to do so until federal disaster assistance is received after the great floods.

Sand and Gravel Mining

Mining of sand and gravel from riverbeds must be closely regulated to be sure that the river regime is not unreasonably disturbed (e.g., by headcutting, levee undermining, or severe reduction of sand flux to the beach--see the paper by Vito A. Vanoni, Robert Born, and Hasan Nouri in Section 4). On the other hand, sand and gravel operators can help by removing unwanted sand and gravel from reservoirs and improved flood channels, although it may cost more than digging a large pit in a river bottom. Different institutional arrangements could well be used to encourage operators to use more surplus sediments and fewer riverbed excavations.

Flood Hazard Zoning and Proper Hillside Development Ordinances

Ordinances to control development are certainly worthy preventive measures, but they generally are used much too little. Ordinances to control developments in identified flood hazard areas that incorporate the federal requirements of FEMA would prevent or reduce damages from floods up to a 100-year flood.

As discussed above, the City of Los Angeles has adopted successful codes for controlling hillside development to prevent landslides. The National Flood Insurance Program strongly seeks to reduce hazards and discourages rebuilding of washed-out structures in the same hazardous locations. Communities must adopt and enforce meaningful hazard mitigation plans in order for their residents to be eligible for flood insurance (see Dale Peterson's paper in Section 8).

Flood Insurance

Flood insurance, administered by FEMA, provides a sharing of risks and pays for damages. The premiums will be based on the claims experienced over many years. The cost of further structural measures can then be compared with the money saved on insurance premiums (i.e., the benefits). The flood insurance program is growing, but the need to prepare maps of hazard zones, especially involving sediment or mudflow damage, has slowed it down.

Better Coordination of Local, State, and Federal Objectives and Activities

Coordination among the various levels of government would lead to improved flood control and faster settlement of intergovernmental transactions, such as for federal disaster assistance to local governments (see the papers by Ronald Robie, Dale Peterson, and Donald Tillman in Section 8).

RECOMMENDATIONS FOR RESEARCH

Although the region avoided a catastrophe of major proportions in the recent flood years, it would be worthwhile to continue research, using these recent flood experiences, on a variety of topics to improve our flood control systems and mitigate hazards. Topics for additional research are listed below.

1. Long-range weather forecasting.

2. Occurrence of cells of especially intense rainfall.

3. Effects of urbanization on flood peaks.

4. Computer programs for better real-time numerical flood forecasting for major rivers, using telemetered data.

5. Real-time determination of optimum reservoir release strategies during a flood.

6. Adequacy of the design criteria for levees, especially for scour protection at the toe.

7. Mechanics of landslides and mudflows, including evaluation of hazards for insurance and mitigation programs.

8. Detailed case studies of rainfall, runoff, and debris flow for selected small canyons in the San Gabriel and Santa Monica mountains in order to understand the responses of small watersheds better and to help assess risks on alluvial fans, manage the watersheds, and operate (or design) debris basins.

9. Controlled burning of small portions of watersheds on a rotating schedule as a means to reduce the severity of wildfires and ensuing floods and debris flows.

10. Techniques to control bed and bank erosion in streams with erodible beds when they are "starved" for sediment.

11. Benefits and costs of various combinations of structural and nonstructural components of an overall system for reducing damage, loss of life, and personal injury and for sharing the residual risks through insurance and disaster relief.

12. Governmental institutions and regulations needed to reduce hazards and future damages through mapping of areas subject to flooding, debris flows, and landslides and through controlling developments in these areas.

REFERENCES

Brown, William M., and Taylor, Brent D. (1981) "Inland Control Structures," Part D1 of <u>Sediment Management for Southern California Mountains, Coastal Plains and Shorelines</u>, Report 17-D1, Environmental Quality Laboratory, California Institute of Technology, Pasadena, 28 pp.

Brownlie, William R., and Taylor, Brent D. (1981) "Coastal Sediment Delivery by Major Rivers in Southern California," Part C of <u>Sediment Management for</u>

Southern California Mountains, Coastal Plains and Shoreline, Report 17-C, Environmental Quality Laboratory, California Institute of Technology, Pasadena, 314 pp.

California Department of Water Resources (1980) "California Flood Management: An Evaluation of Flood Damage Prevention Programs," Bulletin 199, California Department of Water Resources, Sacramento, 277 pp.

Campbell, R. H. (1975) "Soil Slips, Debris Flows, and Rainstorms in the Santa Monica Mountains and Vicinity, Southern California," USGS Professional Paper 851, U.S. Geological Survey, Reston, Virginia.

Fall, Edward E. (1981) "Regional Geological History," Part A of Sediment Management for Southern California Mountains, Coastal Plains and Shoreline, Report 17-A, Environmental Quality Laboratory, California Institute of Technology, Pasadena, 33 pp. plus 3 maps.

Kolker, O. (1981) "Inland Artificial Sediment Movements," Part D2 of Sediment Management for Southern California Mountains, Coastal Plains and Shoreline, Report 17-D2, Environmental Quality Laboratory, California Institute of Technology, Pasadena, 24 pp.

National Research Council (1982) "Selecting a Methodology for Delineating Mudslide Hazard Areas for the National Flood Insurance Program," Advisory Board on the Built Environment, National Academy Press, Washington, D.C.

Taylor, Brent D. (1981) "Inland Sediment Movements by Natural Process," Part B of Sediment Management for Southern California Mountains, Coastal Plains and Shoreline, Report 17-B, Environmental Quality Laboratory, California Institute of Technology, Pasadena, 88 pp.

Troxell, Harold, et al. (1954) "Hydrology of the San Bernardino and Eastern San Gabriel Mountains," Hydrologic Investigations Atlas HA1, U.S. Geological Survey, Reston, Virginia.

Wells, Wade G. (1981) "Effects of Fire on Sedimentation Processes," Part D4 of Sediment Management for Southern California Mountains, Coastal Plains and Shoreline, Report 17-D4, Environmental Quality Laboratory, California Institute of Technology, Pasadena, 40 pp.

Wells, Wade G., and Palmer, Nancy R. (1981) "Role of Vegetation in Sedimentation Processes of Coastal Southern California," Part D3 of Sediment Management for Southern California Mountains, Coastal Plains and Shoreline, Report 17-D3, Environmental Quality Laboratory, California Institute of Technology, Pasadena, 20 pp.

METEOROLOGIC AND OCEANOGRAPHIC CONDITIONS FOR THE ENHANCEMENT OR SUPPRESSION OF WINTER RAINS OVER CALIFORNIA

by Jerome Namias

This paper describes some coupled atmosphere-ocean systems that have led to excessive or deficient winter rains over much of California. These systems are slow changing and of large scale, often embracing the great North Pacific gyres and much of the North Pacific, and involve the North Pacific anticyclone, the Aleutian low, and the associated jet streams and long waves in the upper westerlies. In different winters the atmospheric systems usually depart from their normal position and orientation, so that the character of the air masses deployed by them changes markedly between winters. In turn these wind and weather systems alter the extraction of heat from the ocean and its vertical and horizontal advection, thus resulting in substantial sea surface temperature (SST) anomalies. These anomalies often penetrate to a couple of hundred meters and span areas as large as one fourth of the North Pacific. Therefore they constitute vast heat reservoirs for the overlying atmosphere. Since the time constant of the upper-layer oceanic thermal patterns is an order of magnitude slower than the overlying weather patterns, the signature of an abnormal ocean often lasts for months, seasons, and sometimes years.

With the proper reinforcement from atmospheric patterns, these oceanic thermal reservoirs can feed back to maintain and amplify an abnormal atmospheric pattern. This kind of interaction seems to have taken place strongly during the winters of 1980, 1978, and 1969. In fact, the characteristic rain-enhancing SST patterns were first generated in antecedent seasons. Details of these cases are demonstrated with the help of synoptic charts showing atmospheric and SST fields.

Jerome Namias is Research Meteorologist with the Scripps Institution of Oceanography in La Jolla, California.

Note: My thanks go to Daniel Cayan for editorial assistance, Madge Sullivan for computational assistance, Fred Crowe and his group for drafting, and Carolyn Heintskill for typing the manuscript. This research was supported by the National Science Foundation's Office of Climate Dynamics under Contract No. ATM79-19237, the National Oceanic and Atmospheric Administration under Contract No. NOAA04-8-M01-188, and the Department of Water Resources under Contract No. SDWR-B53326.

INTRODUCTION

The precipitation regime over California is extremely complex. This complexity is associated with great geographic differences in the state, including coastal and orographic influences, seasonal variations, and other factors. Superimposed on this spatial and regular seasonal variability are surprisingly large interannual variations in precipitation. While there is no simple explanation for these interannual variations, research over the past decade indicates that they are associated with large-scale (almost hemispheric) aberrations in the general circulation of the atmosphere and with concomitant variations in the thermal characteristics in the upper layers of the ocean--perhaps the first 300 to 400 m (see Bjerknes, 1969; Namias, 1975; Newell, 1979). This paper presents evidence that the coupled ocean-atmosphere system modulates rainfall over much of California. In this vein some suggestions will be given for the enhancement or suppression of recent winter rains.

MACROSCALE PHYSICAL PROCESSES ASSOCIATED WITH CALIFORNIA PRECIPITATION

As with most temperate-latitude areas, precipitation in California is primarily associated with ascending air motion while dryness is associated with subsiding air. Ascending air currents produce the well-known adiabatic cooling of air, while subsiding motions result in adiabatic heating. After condensation is reached in ascending air, latent heat is liberated and the vertical speed of ascending parcels often increases by two orders of magnitude--from centimeters per second to meters per second. A central question arises as to how these ascending and descending air motions are brought about. Obviously, one effect is introduced by orographic lifting of moist air from the Pacific by coastlines and mountains. Occasionally, air from the continent is forced westward down the mountains, resulting in descending motion and suppressed precipitation. One could estimate the amount of precipitation by knowing the component of air flow from the Pacific onto California. An additional influence is the complicating vertical motions introduced by cyclonic storms, for it is along the fronts of these storms that the strongest vertical motions are usually found. A good deal of California's and the West Coast's precipitation falls from unstable air masses flowing into these cyclones. If these air masses are statically unstable (rapid decline of temperature with elevation), ascending motion takes place in convective cells that can give rise to intense showers. Strong inversions or marked vertical stability makes substantial precipitation less likely. This situation is frequently associated with anticyclonic circulations and subsidence, and sometimes leads to drought.

Figure 1 shows the strong dependence of winter rainfall over the southern California coastal area on the pressure distribution in the lower atmosphere. This chart shows that heavy precipitation over southern California is closely related to sea level pressure over the Pacific in the sense that low pressure off the coast of central California, and, indeed, out to about 140°W, is strongly correlated with southern California precipitation. Also, it shows that heavy precipitation over southern California is associated with higher than normal pressure in the area west of about 140°W. These lines of equal

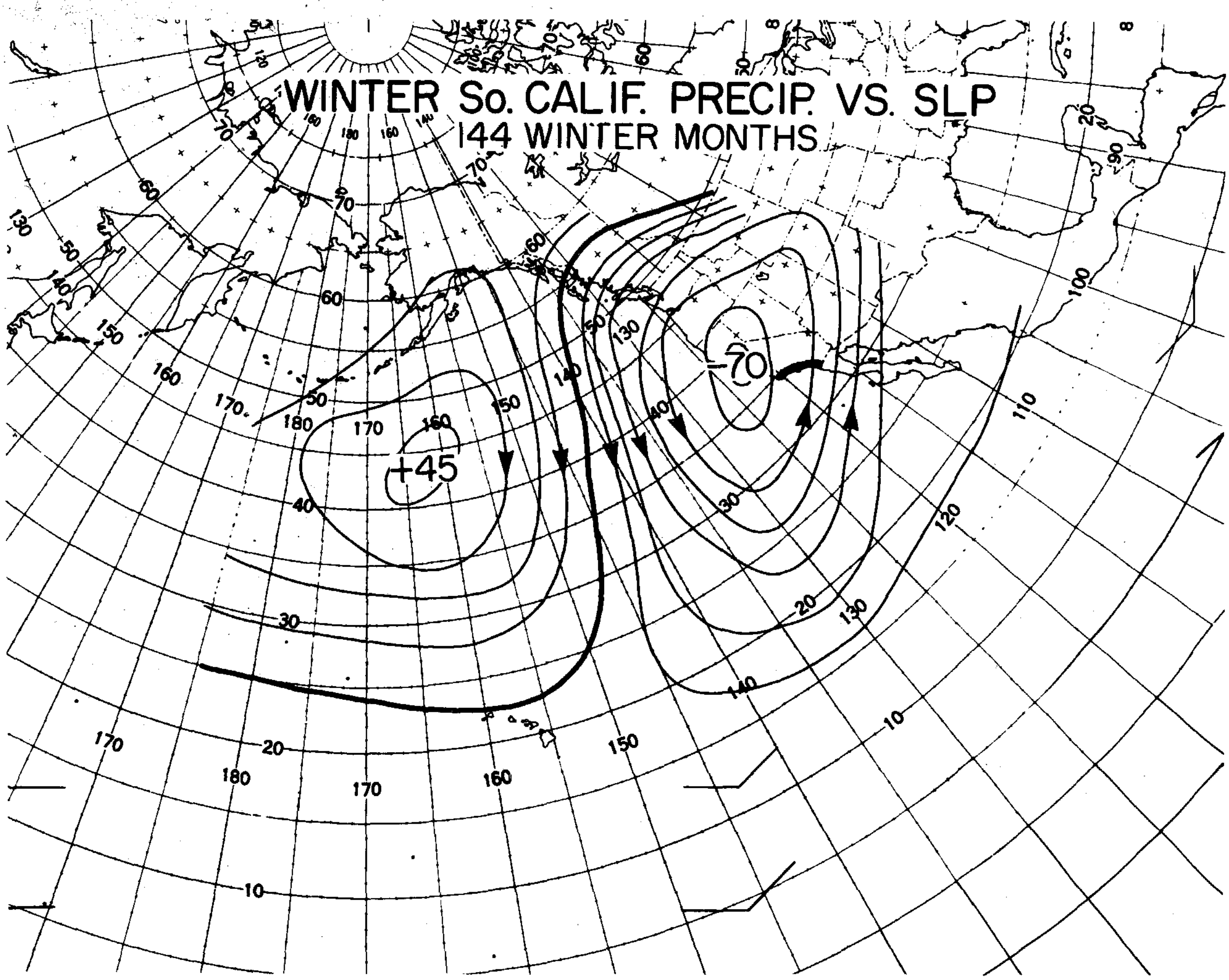


FIGURE 1 Lines of equal correlation (drawn for each 0.10 with centers labeled) between precipitation in southern California (heavy shaded area) and sea level pressure (SLP) elsewhere as computed from 144 winter months (Decembers, Januarys, and Februarys). Arrows show anomalous component of geostrophic wind flow when precipitation is heavy.

correlation can also be associated with prevailing flow in the lower atmosphere, as shown by Stidd (1954). Thus Figure 1 indicates that the optimum case for heavy precipitation involves an anomalously strong southwesterly component of air flow from the offshore coastal waters and an anomalously strong north to south component in the remainder of the eastern Pacific. Since the northerly component usually brings cold air and the southwesterly component brings warm air, the figure suggests that appreciable temperature contrast (baroclinicity) is associated with the storms. It also suggests that the orographic effects operating on the southwesterly air stream are ideally placed to enhance ascending motion. If, on the other hand, the -0.70 correlation area is associated with high pressure, then reverse arrows *would show an anomalous anticyclonic system with descending motion (Santa Ana* conditions) in southern California. In this case low-pressure systems would lie in the domain of positive correlations in the east-central Pacific.

In most cases of abnormally wet or abnormally dry winters, the concept just stated can be verified. From a pragmatic standpoint of long-range forecasting, these concepts stress that one must first predict the anomalous flow pattern in the atmosphere before hoping to estimate the precipitation distribution.

This problem, essentially one of causality, will be addressed in the subsequent sections.

SPACE AND TIME SCALES OF ATMOSPHERIC AND OCEANIC TIME-AVERAGED SYSTEMS

The spatial patterns of sea surface temperature (SST) and 700 mb height (or sea level pressure) are highly coherent. For example, Figure 2 shows the space scales of 700 mb height and sea surface temperature for the North Pacific, with values at each 5^{o} intersection correlated with the central diamond values. This figure shows the high spatial coherence in both air and sea patterns and the similarity of the correlation fields for these elements. The characteristic dimensions of both air and sea anomalies are roughly one third to one half the size of the North Pacific. This coherence is largely *the result of large-scale effects of long waves (Rossby waves) in the* atmosphere, which are closely coupled to the underlying SST patterns. The coupling results from several, often synergistic, factors that come into play when cold, dry air currents extract large amounts of sensible and latent heat from the ocean and create regions of upwelling (in cyclones) and downwelling (in anticyclones). Anomalous surface Ekman drifts of warm or cold water also contribute to produce anomalies. We shall have more to say about this coupling later on.

As for time scales, a striking indication of the differing time scales in the ocean and the atmosphere is afforded by Figure 3. The patterns of sea surface temperature, sea level pressure, and 700 mb height were autocorrelated at discrete lags of 1 to 12 months by using standardized anomalies computed for each 5^{o} square for each month of each of the 20 years 1947-66. The resulting correlations give a measure of the degree of similarity of the anomaly patterns for one month to any other month. Figure 3 shows that the upper ocean retains its anomalous temperature pattern for a much longer time than the atmosphere retains its pressure pattern, and thus the ocean can provide a heat storage memory to influence the overlying atmosphere at a later date. To a large extent this strong pattern autocorrelation in the sea is a result of the high specific heat of water, the substantial depth of anomalies (often below 200 m), and slow ocean currents.

The most important thing to note in this figure is the long time constant for SST patterns relative to 700 mb contour patterns. This difference means that the ocean, having a long time scale, could serve as a reservoir or memory for the relatively fast-changing atmosphere. That is, once the upper thermal structure is disturbed relative to normal, it might retain this abnormal

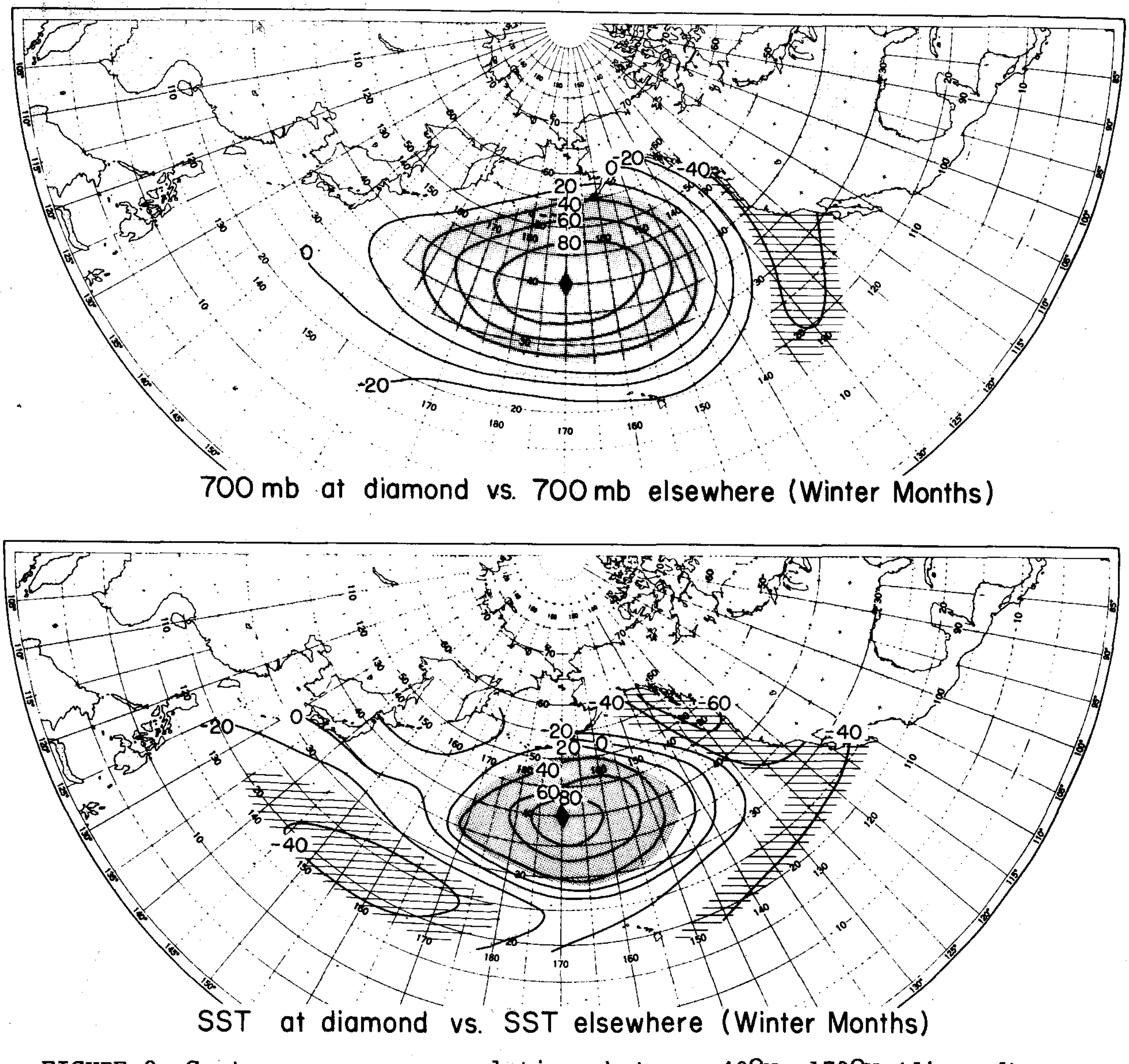


FIGURE 2 Contemporaneous correlations between 40°N, 170°W (diamond), and elsewhere for 700 mb heights (upper) and sea surface temperature (lower). Values are computed from about 20 years of monthly mean data. Shaded areas represent correlations exceeding the one percent level of significance, with positive correlation stippled and negative correlation patched.

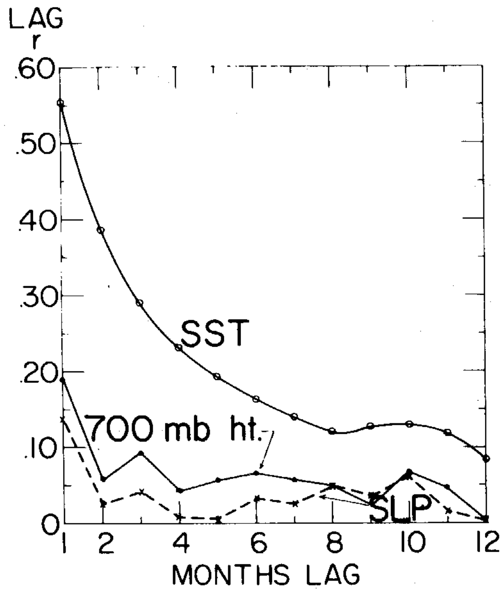


FIGURE 3 Overall autocorrelations of standardized values of monthly mean sea surface temperature, 700 mb height, and sea level pressure determined from a 5° grid of points covering the North Pacific (north of 20°N) during the 20-year period 1947-66.

pattern for several months and thereby encourage the atmosphere to return to an abnormal track. We shall give illustrations of this phenomenon in this report.

SOME AIR-SEA FACTORS ASSOCIATED WITH RECENT WET WINTERS

The extreme precipitation observed in California during the winter of 1979-80 was associated with extreme patterns of both atmospheric temperature and sea surface temperature. Figure 4 shows in its lower portion the observed anomalous components of 700 mb height and the associated SST patterns for this winter. The very large negative departures from normal over much of the eastern Pacific imply frequent and intense storminess in an area normally dominated by the North Pacific anticyclone. It also shows that the air influencing much of California frequently arrived with a southerly component, indicating the advection of tropical air. (More about the characteristics of the individual storms is presented in subsequent papers.) The view in Figure 4 is the entire winter mean and thus only the statistical aggregates of the storms. Coupled to the highly anomalous atmosphere is the anomalous SST pattern over the eastern Pacific. A strong gradient of sea surface temperature appears between the warm eastern Pacific and the cold water to its west. The theory proposed is that not only did the atmosphere influence the sea but, equally important, the sea influenced the overlying atmosphere by encouraging the development of storms in the contrasting area between the cold and warm water. This implies that the water imparted its heat rapidly to the overlying air and produced a zone of enhanced baroclinicity on which storms could feed.

It has been shown in many papers (see, e.g., Namias, 1975) that the contemporaneous coupling between air and sea can be captured qualitatively by the use of stepwise multiple regression. In this case one might ask the following questions:

1. Given the atmospheric anomalies in 700 mb height distribution indicated in the lower half of Figure 4, what would be the associated SST pattern?
2. Given the SST pattern observed in the lower part of Figure 4, what would be the associated 700 mb anomalous pattern?

Both these questions are answered in the top part of the figure (labeled "specified"). From these specifications it is seen that both the atmospheric patterns and sea surface temperatures are similar though not precisely the same as observed. The conclusion is that <u>if</u> one knew either pattern, the sea surface temperature or the 700 mb height, a good estimate of the other pattern could be made. Because of the longevity of the sea surface temperature relative to the upper-level contour patterns, it might be easier to predict the SST pattern. In this case, as in others studied, the SST patterns were indeed remarkably similar to one another not only during the winter months of December, January, and February 1979-80 (Figure 5) but also for the antecedent months, September, October, November, and December 1979 (Figure 6). This series of patterns may mean that the disturbed ocean thermal structure of the fall of 1979 remained roughly unchanged until late winter, when the westerlies and associated storms spread southward. At this time of southernmost displacement, storms could draw upon the enhanced atmospheric baroclinicity provided by the SST contrast.

The fall of 1979 may thus have been one where the underlying ocean offered a clue as to the probability of extreme winter rains. It cannot be overly stressed, however, that in order for this coupling to take place both atmosphere and ocean must collaborate. In this case the collaboration was enhanced because of the seasonal forcing of storm tracks and westerlies southward in winter (particularly late winter), so they could draw upon the reservoirs of energy supplied by the underlying ocean.

It is interesting to note the vertical temperature and humidity distributions for February 1980 and to compare these with the values for 1977--a drought year. These soundings are shown in Figures 7A and 7B for San Diego along with the striking differences in rainfall between the two Februarys. A major difference appears in the static stability of the lowest layers of the atmosphere (below 850 mb). In the wet February of 1980 the low levels were much more unstable and more moist than in the dry year. This circumstance would make it easier for convective rains to occur than in 1977, when there was a much stabler lower layer with frequent inversions and appreciably dryer air.

A similar pattern of sea surface temperature in collaboration with atmospheric developments was observed in the heavy rains of the winter 1968-69. The SST patterns antecedent to that winter are reproduced in Figure 8. These figures show the strong temperature contrast between warm water in the extreme eastern Pacific and cold water to its west.

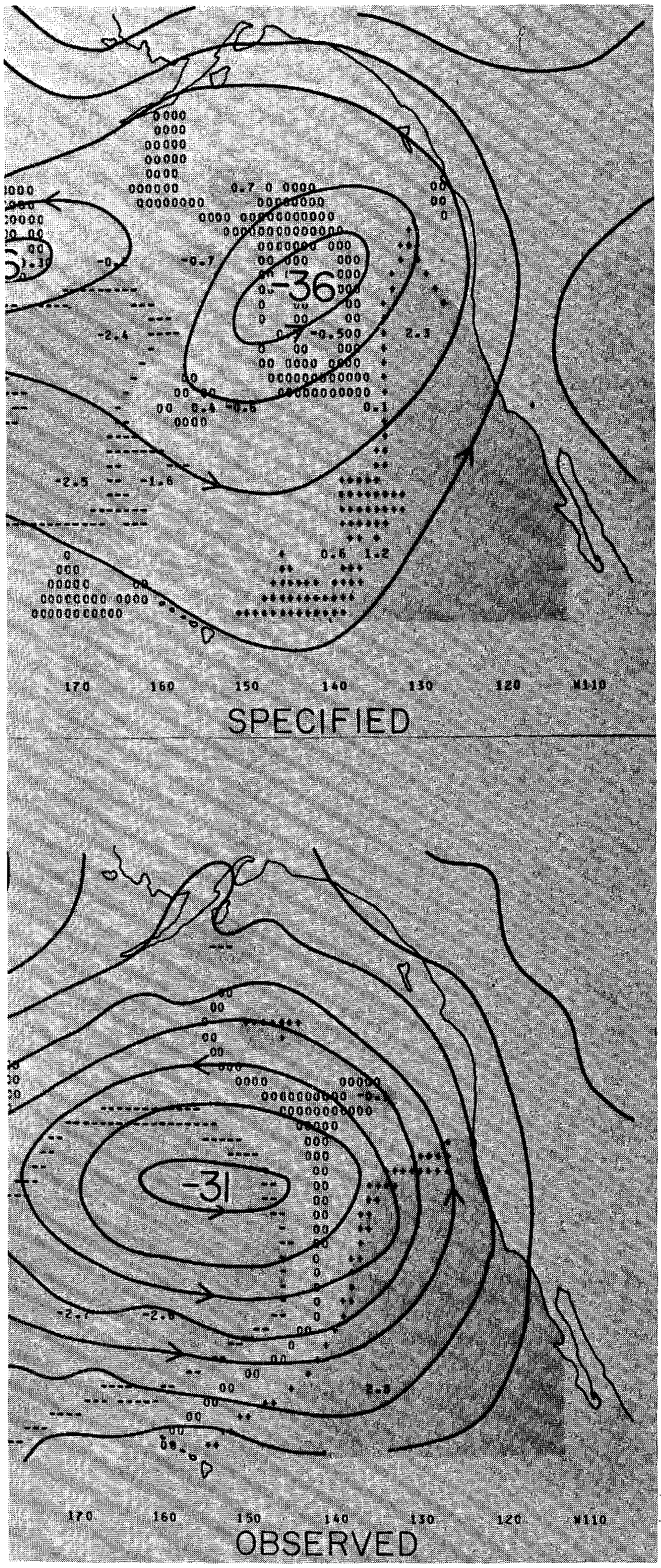


FIGURE 4 Lower: SST anomaly (shaded) observed during the winter of 1979-80 and associated lines of departure from mean (DM) of 700 mb height (in tens of feet). Arrows indicate anomalous flow. Temperature deviations are drawn for intervals of greater than 1°F above normal (+) and for greater than 1°F below normal (-). Upper: The predicted 700 mb anomalies and SST anomalies from observed conditions shown in the lower figure.

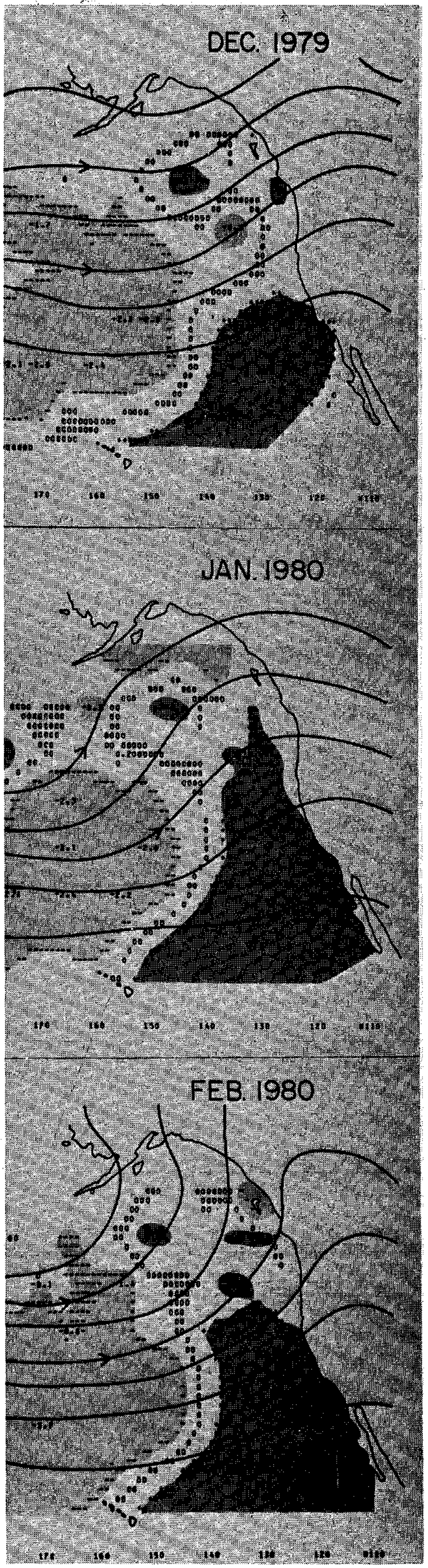


FIGURE 5 Observed SST anomalies (+'s and -'s) and lines of constant 700 mb height for indicated months.

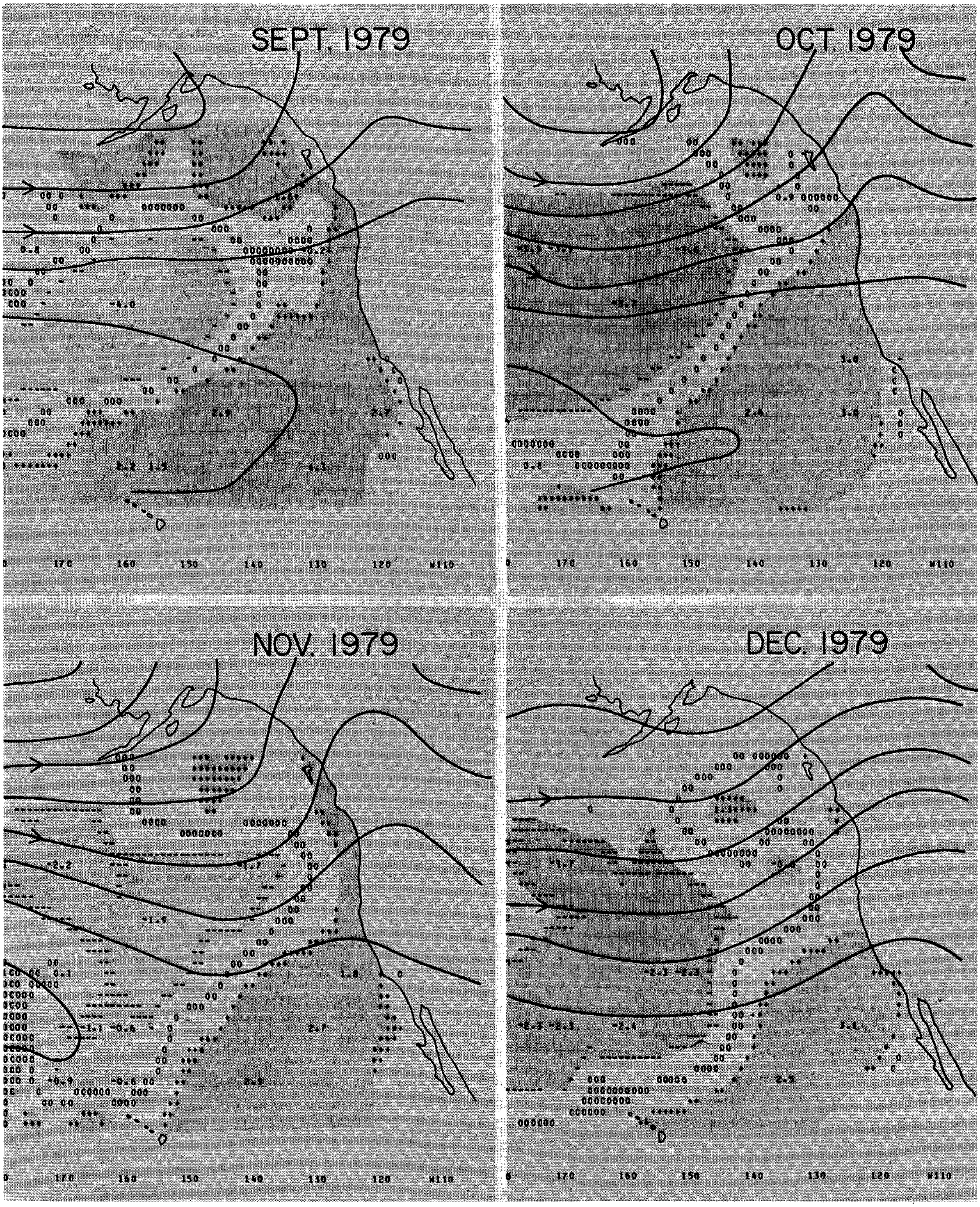


FIGURE 6 Observed SST anomalies (+'s and -'s) and lines of constant 700 mb height for indicated months.

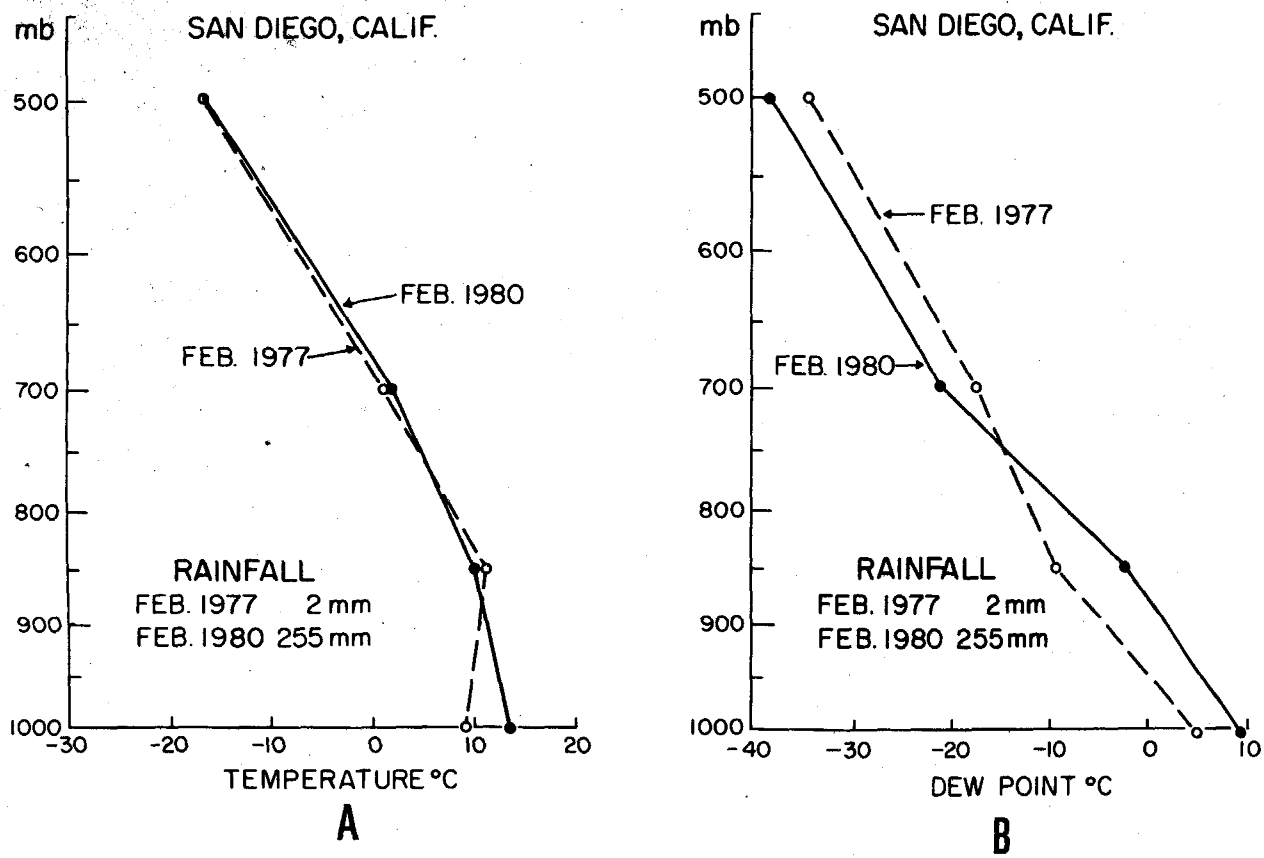


FIGURE 7 Average vertical temperature (A) and dew point distribution (B) for February 1980 (wet) and February 1977 (dry) at San Diego, California, along with associated rainfall in these two months.

CONDITIONS ASSOCIATED WITH THE 1976-77 WINTER DROUGHT AND THE FOLLOWING WET WINTER

As seen from Figure 9, the 1976-77 drought affected a large portion of the West (see Namias, 1978). This was associated with anomalous patterns of sea surface temperature and 700 mb height opposite to those observed in the two wet winters discussed above. The observed conditions of the winter of 1976-77 are shown in Figure 10. The meridionally oriented SST pattern and 700 mb anomalies suggest that storms would be forced northward to Alaska while the West would be dominated by anticyclonic conditions, subsidence, and consequent lack of rains.

Referring to Figure 9, it will be noted that the precipitation pattern changed dramatically between the winters of 1976-77 and 1977-78, when the drought was broken over the entire West (see Namias, 1979). Precursors to this impending break were evident as early as the fall of 1977, when the signs of 700 mb anomalies were almost opposite to those observed in the fall of 1976

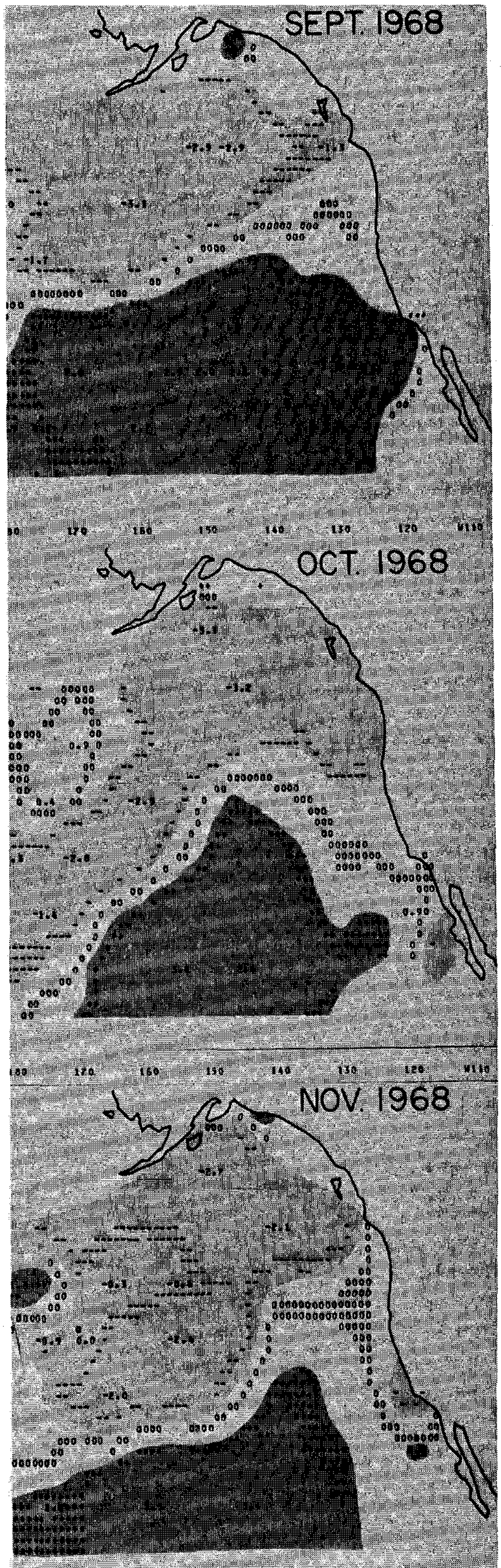


FIGURE 8 SST anomalies observed in the fall months of 1968.

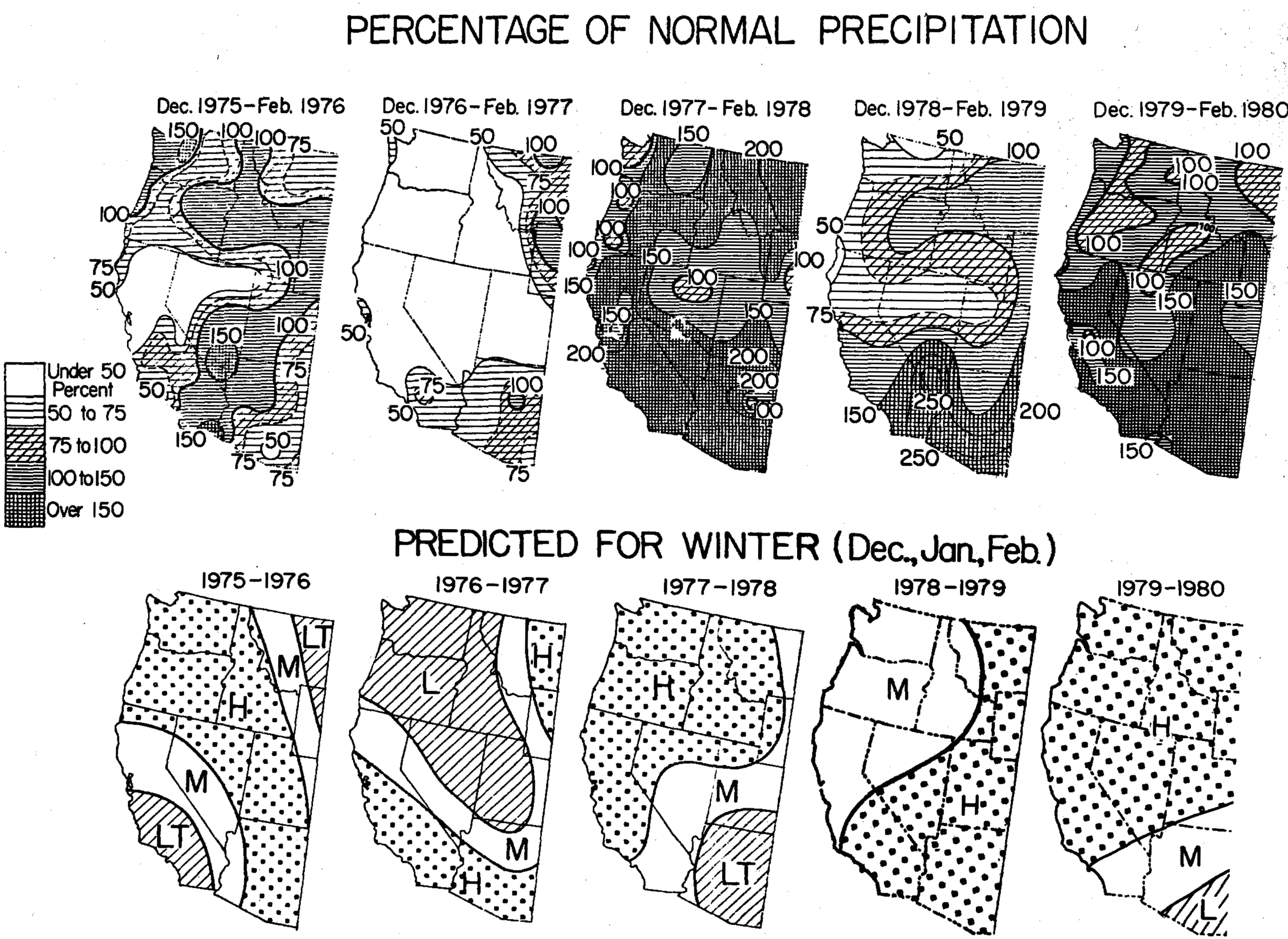


FIGURE 9 Upper: Percentage of normal precipitation over the western third of the United States during the past five winters. Lower: Predicted precipitation in three equally probable classes--light (L or LT), moderate (M), and heavy (H).

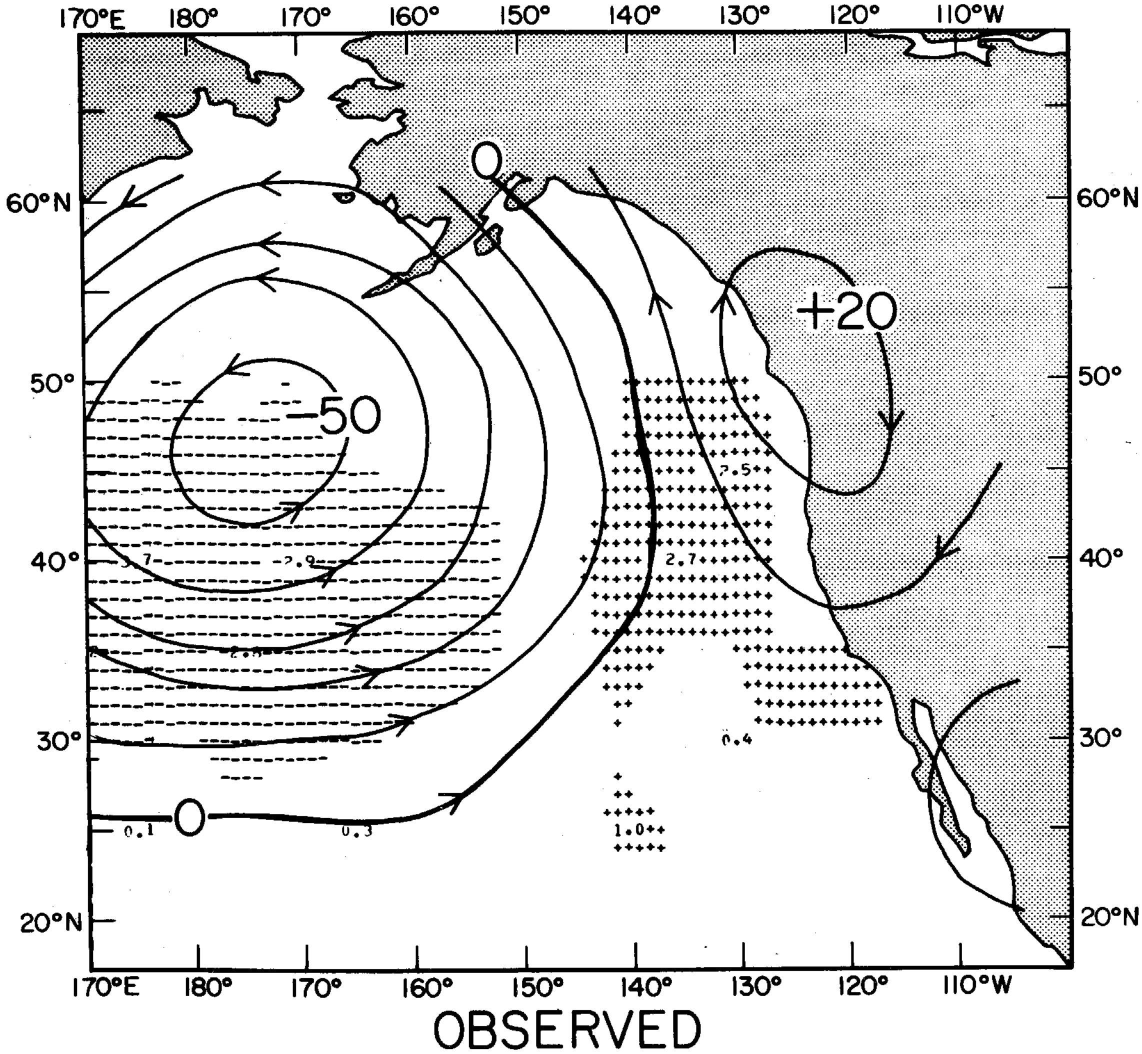


FIGURE 10 Observed SST anomalies (+'s and -'s) and associated 700 mb anomalies (solid lines with arrows) for the winter of 1976-77. Virtually all of the western United States was dry during this winter, in association with this pattern.

over most of the hemisphere (the pattern correlation between these falls was -0.41). Equally important was the fact that the antecedent SST anomaly distribution was also strikingly different. Using the stepwise multiple regression equations mentioned earlier, the probable winter 700 mb anomalies for 1977-78 were constructed at the end of September, October, and November 1977. These prognostic charts are reproduced in Figure 11. Given the SST patterns observed in the fall months of 1977, when advected with normal ocean current movements, the subsequent winter's 700 mb specified pattern has large negative anomalies affecting the West Coast. From material such as shown in Figure 1 and descriptions thereof, we see that these are indeed heavy rain patterns for much of the West. Therefore the break in the drought had

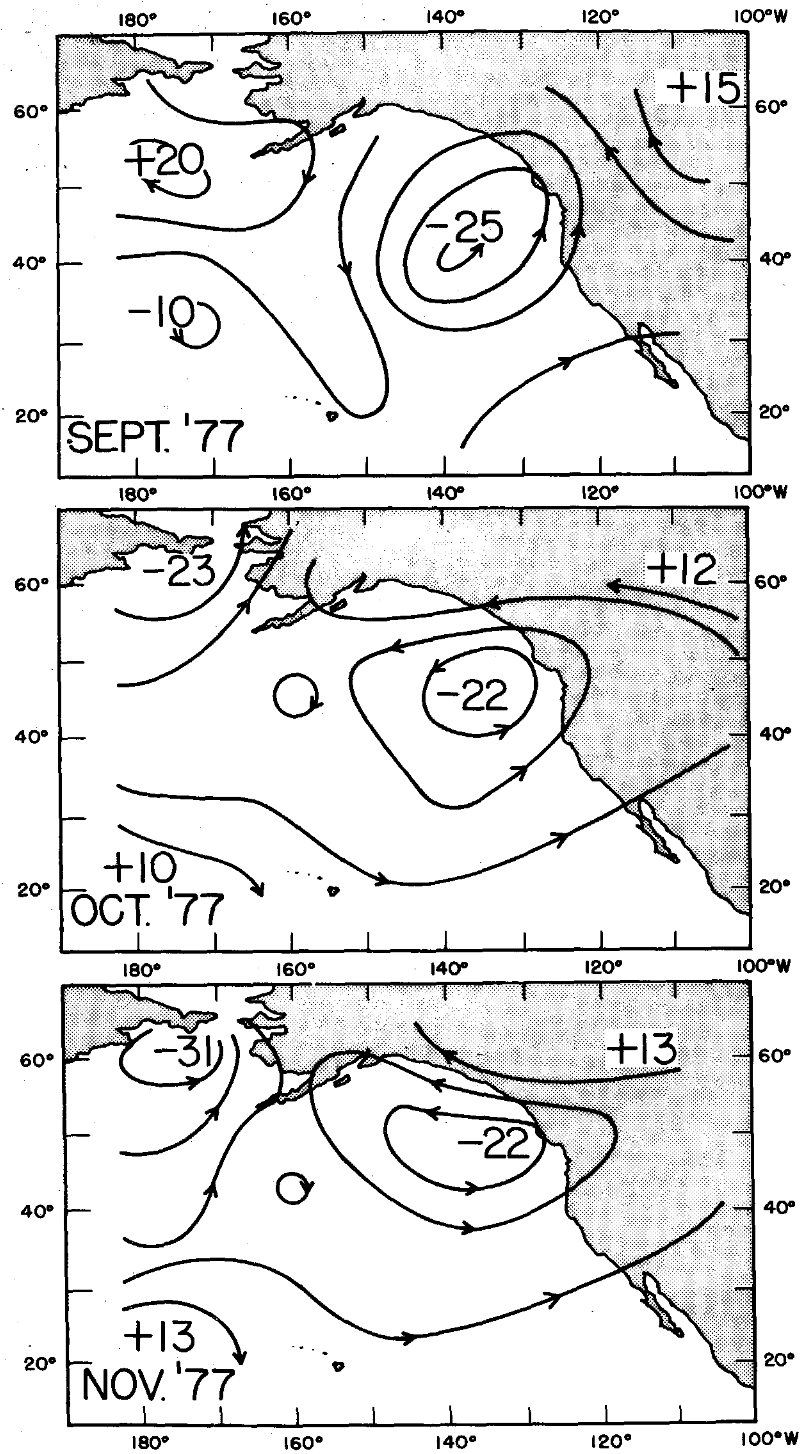


FIGURE 11 Estimated pattern of 700 mb departures from normal for the winter of 1977-78 based on initial conditions and projections made during September, October, and November 1977.

antecedent signs that were strong enough to hazard a prediction of such a break.

The SST pattern observed for the 1978 winter is reproduced in Figure 12, where we again see the contrast between warm water off California and cold water in the central Pacific--a strong gradient that could influence the overlying atmosphere and act to excite storms.

FINAL REMARKS

From the studies described and referenced above, wet and dry winters in California are characterized by strikingly different patterns of 700 mb height and sea surface temperature. The contemporaneous relationship between monthly or seasonal averages of sea surface temperature and 700 mb height is sufficiently good so that a knowledge of one field can lead to a prediction of the other with fair accuracy. Since the atmosphere persists much less than does the SST field, it might be easier to predict the SST anomaly field and then use stepwise multiple regression equations to obtain an estimate of the associated 700 mb field.

However, this would still require success in predicting the SST field. In recent cases of winters with very heavy or very light precipitation, the antecedent patterns of sea surface temperature have appeared several months before the onset of winter. It is possible that these patterns, themselves formed by abnormal atmospheric events, could lie in wait for seasonal changes in storm tracks and upper-level westerlies so as to produce great abnormalities in precipitation in the subsequent winter.

Obviously, such a complex chain of events requires a great deal of further research before highly successful predictions are possible.

REFERENCES

Bjerknes, J. (1969) Atmospheric teleconnections from the equatorial Pacific Monthly Weather Review 97(3):163-172.

Namias, J. (1975) Short Period Climatic Variations, Collected Works of J. Namias 1934 through 1974, 2 Vols., University of California, San Diego, 905 pp.

Namias, J. (1978) Recent drought in California and Western Europe, Reviews of Geophysics and Space Physics 16(3):435-458.

Namias, J. (1979) Premonitory signs of the 1978 break in the West Coast drought, Monthly Weather Review 107(12):1675-1681.

Newell, R. E. (1979) Climate and the ocean, American Scientist 67:405-416.

Stidd, C. K. (1954) The use of correlation fields in relating precipitation to circulation, Journal of Meteorology 11(3):202-213.

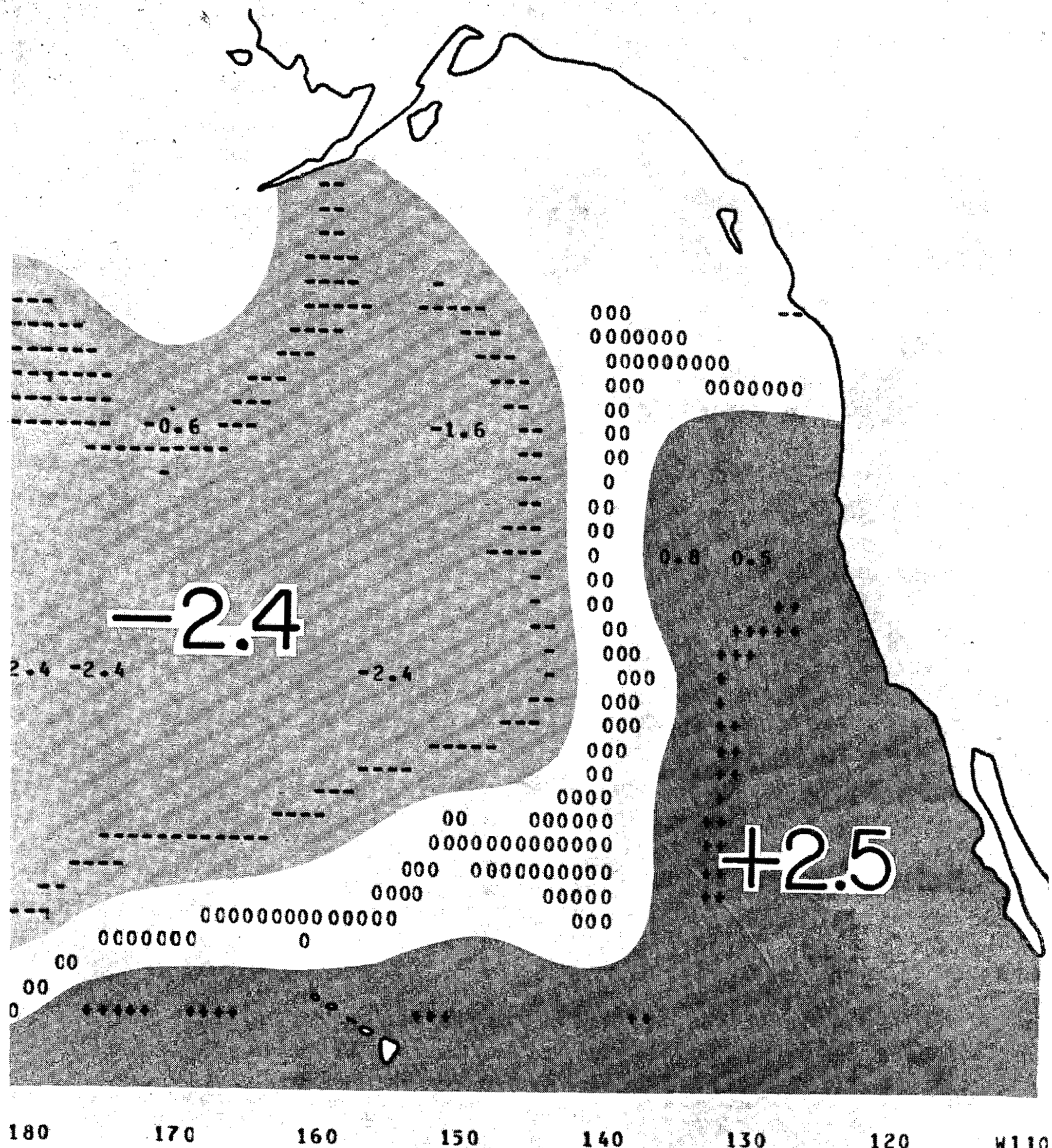


FIGURE 12 SST anomaly pattern observed during the winter of 1977-78. (This pattern was associated with a wet western United States.)

DAMAGE-PRODUCING WINTER STORMS OF 1978 AND 1980 IN SOUTHERN CALIFORNIA: A SYNOPTIC VIEW

by Carlos Garza and Craig Peterson

The winter storms of 1977-78 and 1979-80 produced record rainfall and subsequent property damage and loss of life. However, the storm systems, from a synoptic point of view, were quite different. Although there were many storms during the 1977-78 winter, the storms of February 8-10 and February 28-March 5 produced the most rainfall and the greatest amount of damage. The first storm is seen as a case of strong, rapid development that reached its maximum intensity as it moved over southern California. A strong positive vortex imposed upon a frontal zone is shown to be the reason for strong cyclogenesis. This, combined with a jet stream that had been displaced to low latitudes, resulted in a large precipitation event over southern California. The latter storms of 1978 and also the ones in 1980 were a series of moderate-intensity storms rather than a single large storm. In the days preceding the storms a large high-latitude block formed over the Bering Sea or over Alaska. This block produced a very stable long-wave pattern, allowing the storm track to remain at low latitudes for several days. The 1978 and 1980 storms are discussed synoptically using 500 mb and surface analyses, satellite imagery, and isohyetal analyses.

INTRODUCTION

The winter seasons of 1977-78 and 1979-80 both produced rainfall far greater than average. Rainfall rates on all time scales approached--and in

---

Carlos Garza is Deputy Meteorologist-in-Charge and Craig Peterson is Meteorologist Intern with the National Weather Service in Los Angeles, California.

Note: The authors wish to thank Sylvia Graff for her excellent technical assistance and photographic expertise. Ms. Graff dedicated much of her own time to prepare slides and prints that were used in the presentation. In addition, as the Disaster Preparedness Meteorologist for the Los Angeles office of the National Weather Service, she proved to be an excellent source of information on the researched storms. The authors are also appreciative of Alice Kelleher's time and effort in preparing the initial and final draft for the presentation.

some cases exceeded--past records. Since it would be impossible to describe synoptically the entire winter seasons of 1977-78 and 1979-80, this paper will focus its attention on what the authors feel were the most damaging periods of those seasons.

WINTER STORMS OF 1977-78

During the 1977-78 winter season two storm periods produced vast amounts of damage in southern California. The first disastrous storm occurred during February 8-10, although this was imbedded in a rainy period that extended over February 5-14. Heavy rains falling over the southern San Joaquin Valley and the Los Angeles basin and the surrounding mountains caused nearly $100 million worth of damage (including approximately $43 million in Los Angeles and over $40 million in southern San Joaquin Valley) and were responsible for 20 deaths. The floods, flash floods, and mudslides resulted from rainfall totals that exceeded 16 in. at some mountain stations (for example, Lytle Creek had 16.40 in. and Crystal Lake had 16.53 in.). The isohyetal analysis is shown in Figure 1. While the extensive damage was caused by the heavy rains during this period, other rains during the preceding two months had already saturated the soil and thus also contributed largely to the extensiveness of the damage.

Normally, during the winter season, a strong upper-level westerly flow can intrude into southern California periodically. However, that type of pattern will last only a few days at a time. In the winter of 1977-78 westerlies remained further south than normal for longer time periods. The result was a southerly shift in the mean storm track and therefore an abnormal amount of rains in southern California. During the latter part of January and the early part of February, a fairly stable situation existed, with a well-established upper-level ridge through the western part of the United States through northwestern Canada. Figure 2 depicts a surface pattern that was typical of what occurred during January 15-February 4--the passage of fast-moving frontal systems with only the tail end of these moving through southern California. Most surface lows during that time formed north of 40°N and moved eastward.

By February 5 a dynamic and strong cold-core low, which had been developing over the northern Pacific, continued digging and progressing slowly into the western United States (Figure 3). The jet maximum was located from near Santa Barbara northeast into Idaho. The combination of the intense low and the large amplitude of the midwestern U.S. ridge resulted in a pronounced split of the flow, which was very evident on February 7 (Figure 4). Strong perturbations in the upper-level flow were noted across southern California at this time.

This same type of flow contributed to the unusually strong development of the February 8-10 storm. It was a classic case of frontal wave development. The wave was first evident about 600 miles northeast of Hawaii, and the system moved east-northeast at 40 knots. It intensified at first, then more rapidly as it neared the coast. During the late afternoon of the ninth the storm center moved to a position west of Los Angeles. Then, on the tenth, it moved inland (Figure 5). At 5:54 a.m. (PST) on the tenth the Los Angeles International Airport Weather Office recorded a sea level pressure of 993.7 mb, the lowest ever recorded at that office.

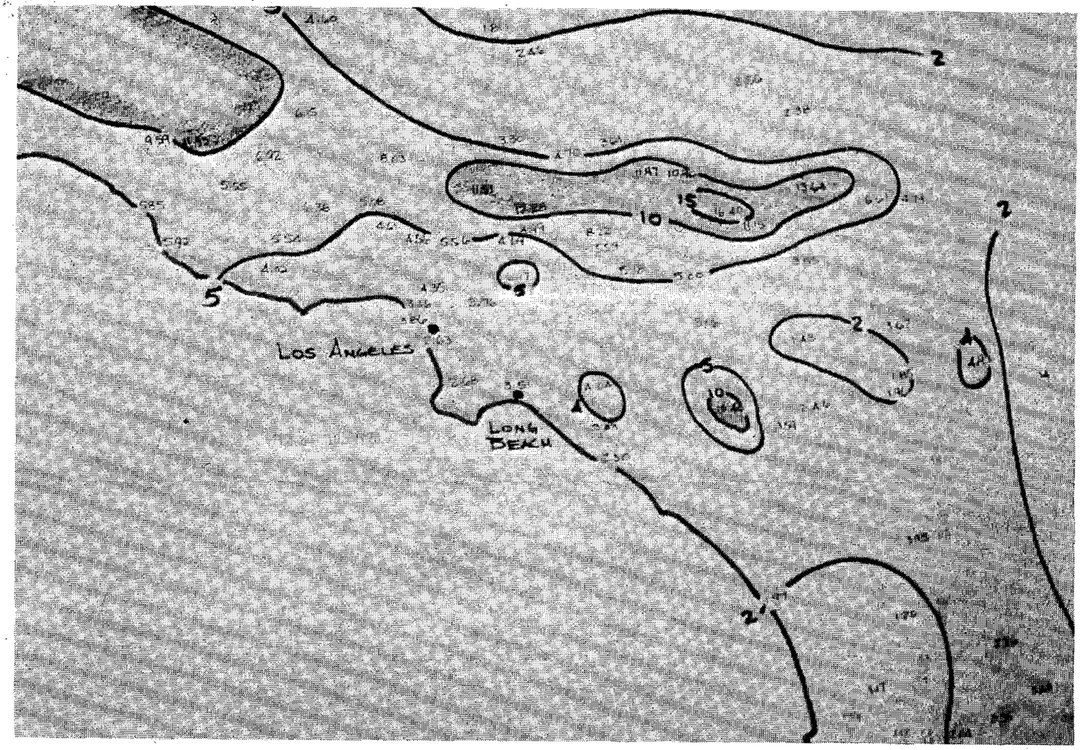


FIGURE 1 Rainfall totals for the storm of February 8-10, 1978.

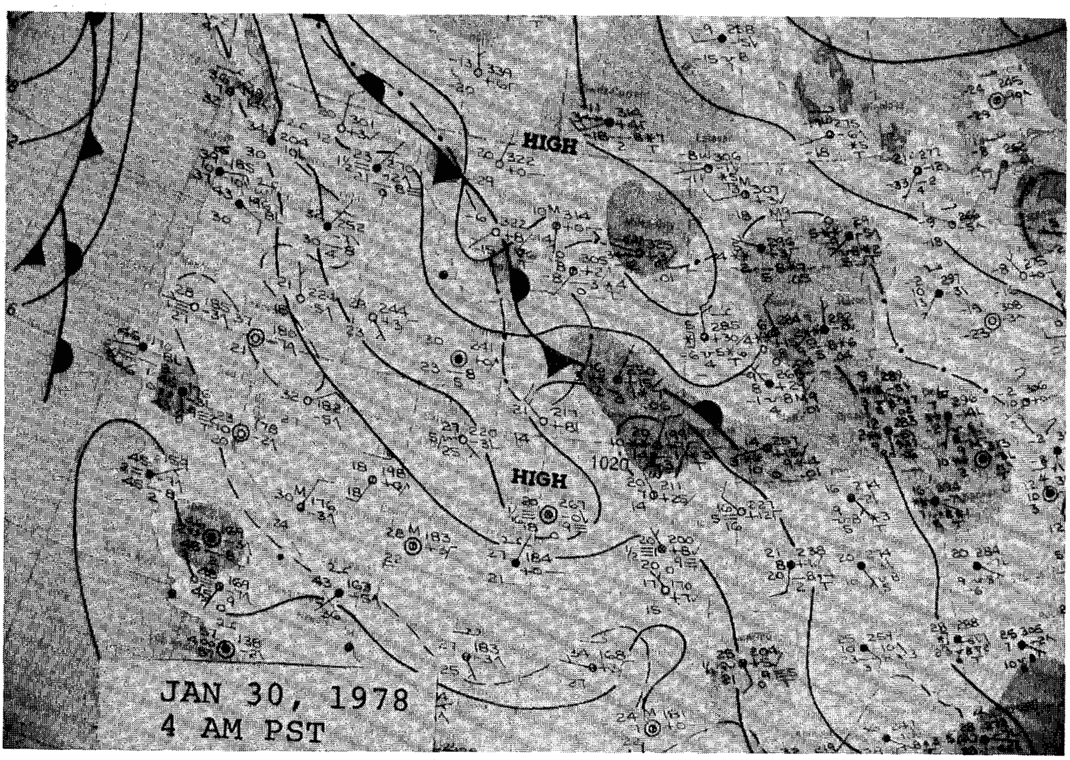


FIGURE 2 Surface weather pattern of January 30, 1978.

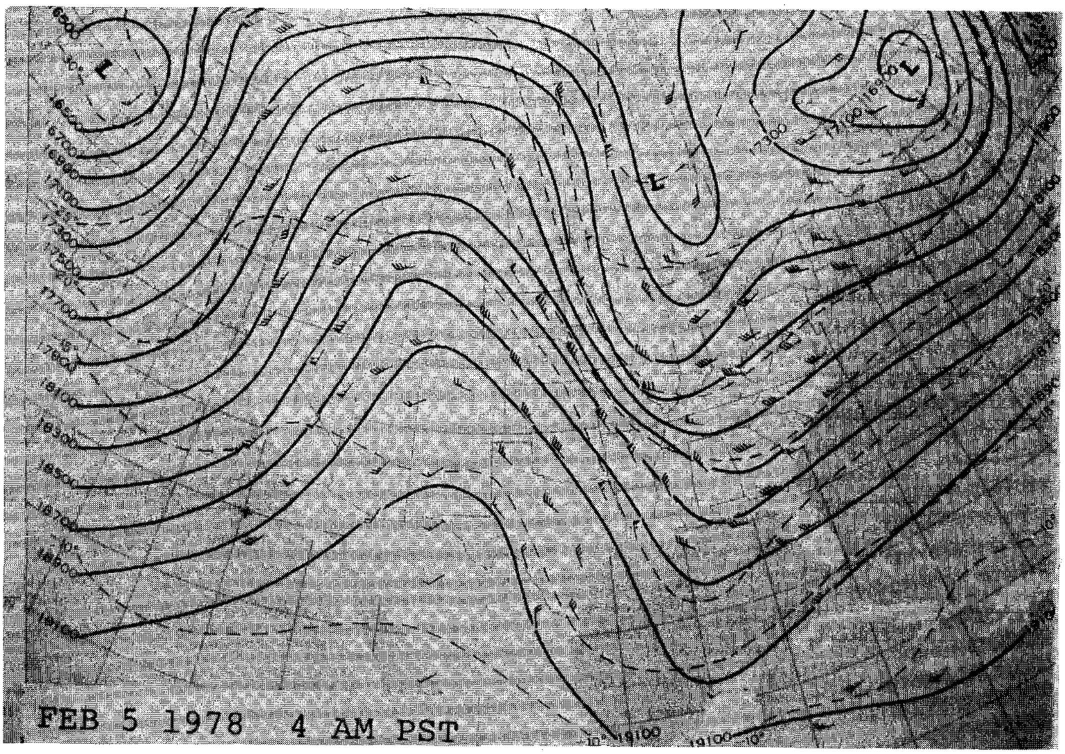


FIGURE 3 500 mb analysis for February 5, 1978.

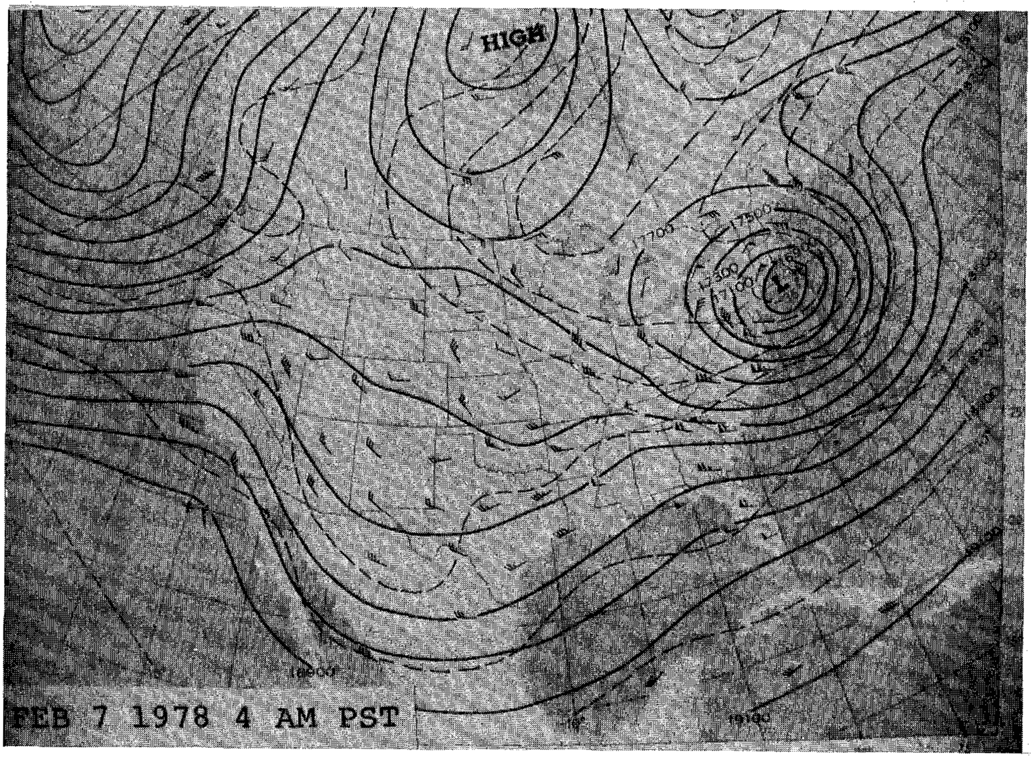


FIGURE 4 500 mb analysis for February 7, 1978.

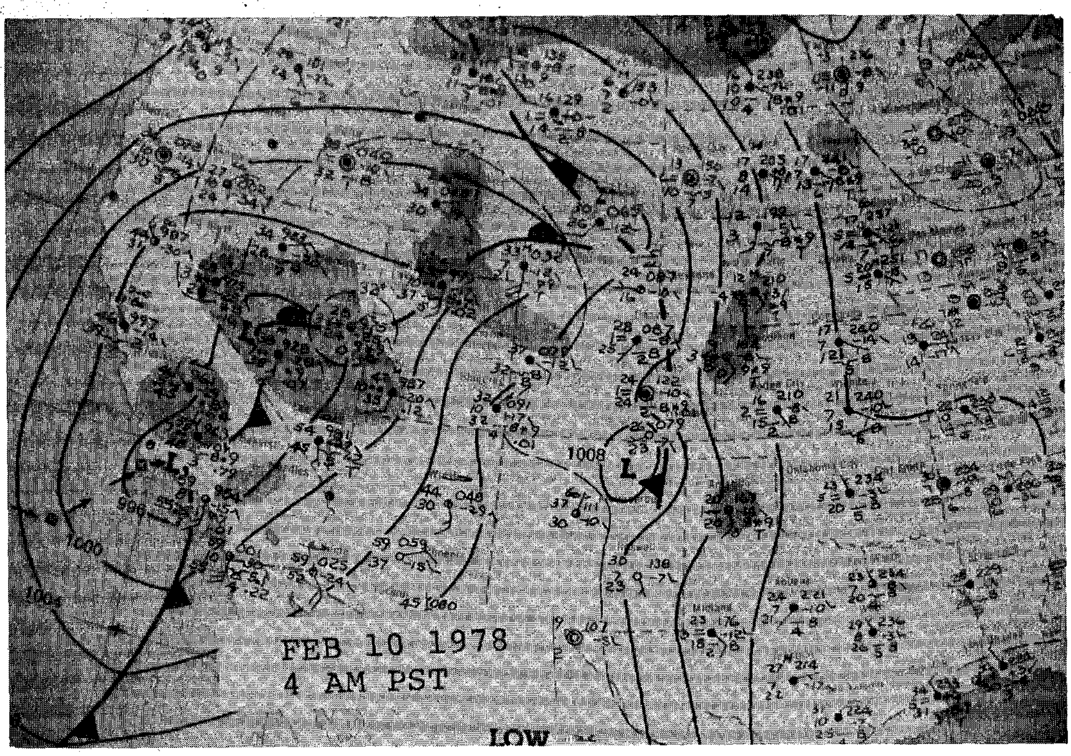


FIGURE 5 Surface weather pattern of February 10, 1978.

Although frontal wave development of this type is not uncommon in the eastern Pacific, it is unusual at such low latitudes. Typically, several storms each year move into southern California from the southwest, but these are usually cutoff upper-level cyclones that weaken as they move inland. The extreme southern extension of the upper-level westerlies during the first part of February set up conditions that were more typical of the Washington or Oregon coasts than of southern California.

There was an important point that was also noted from this storm. Even though there was widespread flooding, some flash flooding, and mudslides, most of the rainfall amounts from this storm were at or below those associated with a 10-year storm. This means that, given the proper antecedent conditions, similar flooding could be expected several times during an average lifetime.

The other significant storm period of the 1977-78 winter season occurred during February 28-March 5, 1978. Storms from this period also had a devastating effect on the Los Angeles basin and surrounding areas. As in the storm of February 8-10, these caused considerable damage and loss of life. Eighteen persons were killed and an estimated $120 million worth of damage occurred. Over 300 homes were damaged in Los Angeles County alone. Figure 6 shows the isohyetal analysis for this period. Rainfall amounts surpassed 20 in. in several locations. Mount Wilson recorded 24.16 in. during that six-day episode.

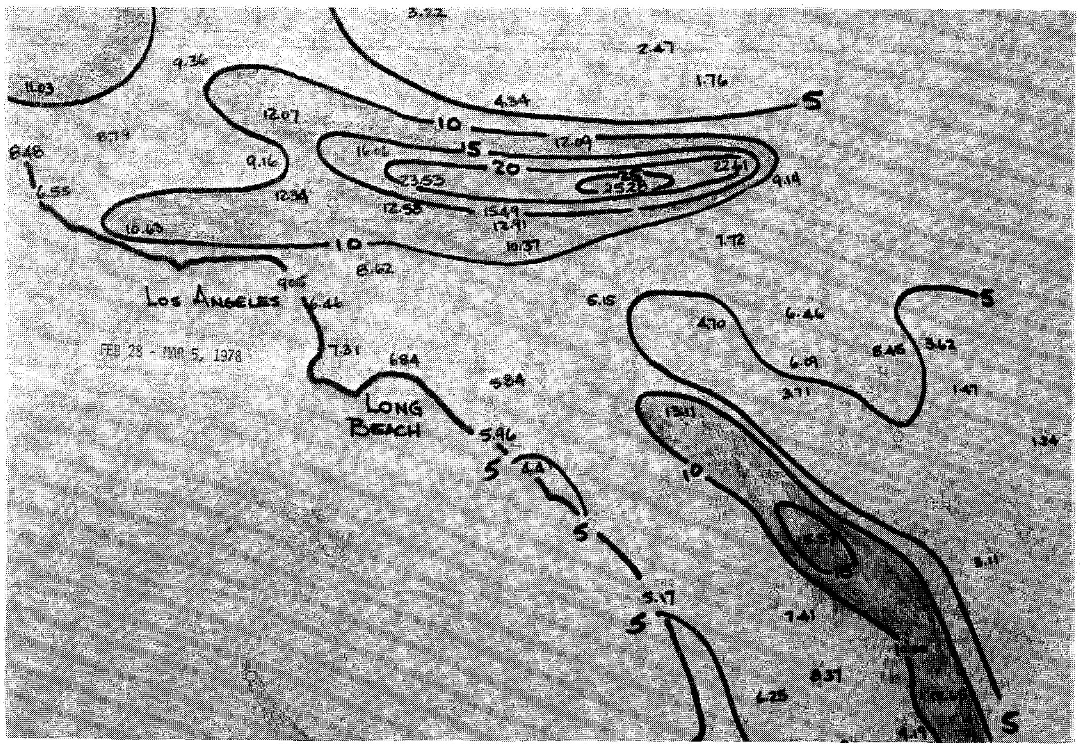


FIGURE 6 Rainfall totals for the storm of February 28-March 5, 1978.

While the earlier storm was developmental in nature, these later storms were the result of another well-known phenomenon--the high-latitude block. Toward the end of the month of February a strong upper-level ridge in the eastern Pacific had been building northward into the Gulf of Alaska. Minor perturbations from a low-pressure system in the Bering Sea would slide over this ridge, causing precipitation to remain well north of southern California. The 500 mb chart for February 26 (Figure 7) depicted the gradual push of the ridge into the Gulf of Alaska. The surface chart for the same date (Figure 8) showed a pattern that had been noted during the preceding two weeks, that is, a fair weather pattern. By the twenty-eighth, however, the upper-level ridge had split, and a well-established block became entrenched over western Canada, with a weaker ridge stretching from Baja northwestward to about 50°N, 145°W (Figure 9). A trough, sliding under the ridge, moved through southern California on the twenty-eighth, and light rains, on the order of 0.5 in. or less, were experienced over most of the southern part of the state. On Wednesday, March 1, the upper-level blocking pattern persisted over Alaska with a low located just off the coast of southern California. A temperature trough was noted just west of the low, indicating continued development. The surface pattern for that period showed an intense surface low (994 mb) moving inland just north of Los Angeles. This low eventually weakened as it moved into Nevada. Another rapidly moving storm entered southern California immediately thereafter. Rainfall totals recorded over the Los Angeles basin during the 24-hour period ending at 7 a.m. Wednesday ranged from 0.75 in. to nearly 1.5 in. The main features during this storm period

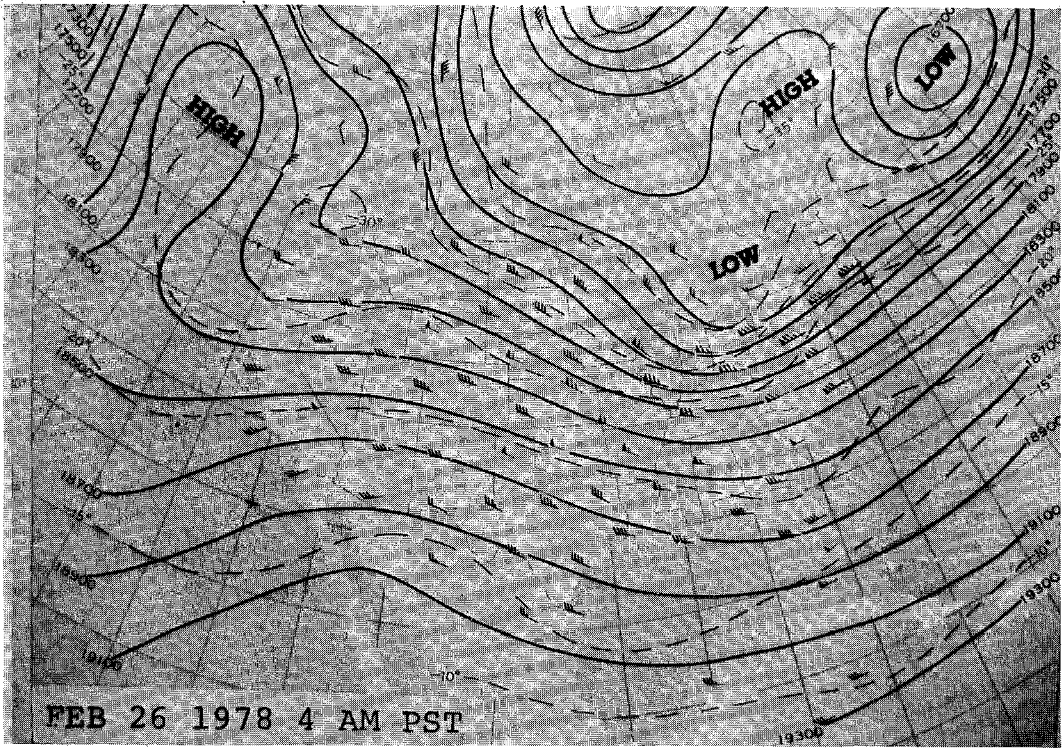


FIGURE 7 500 mb analysis for February 26, 1978.

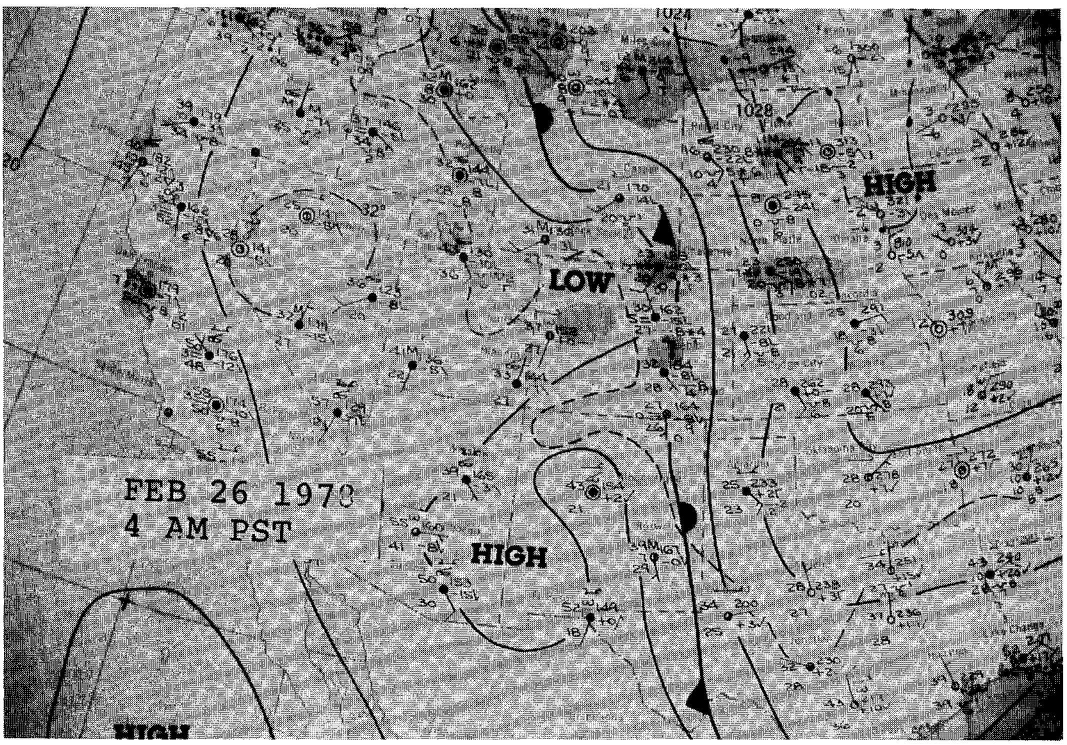


FIGURE 8 Surface weather pattern of February 26, 1978.

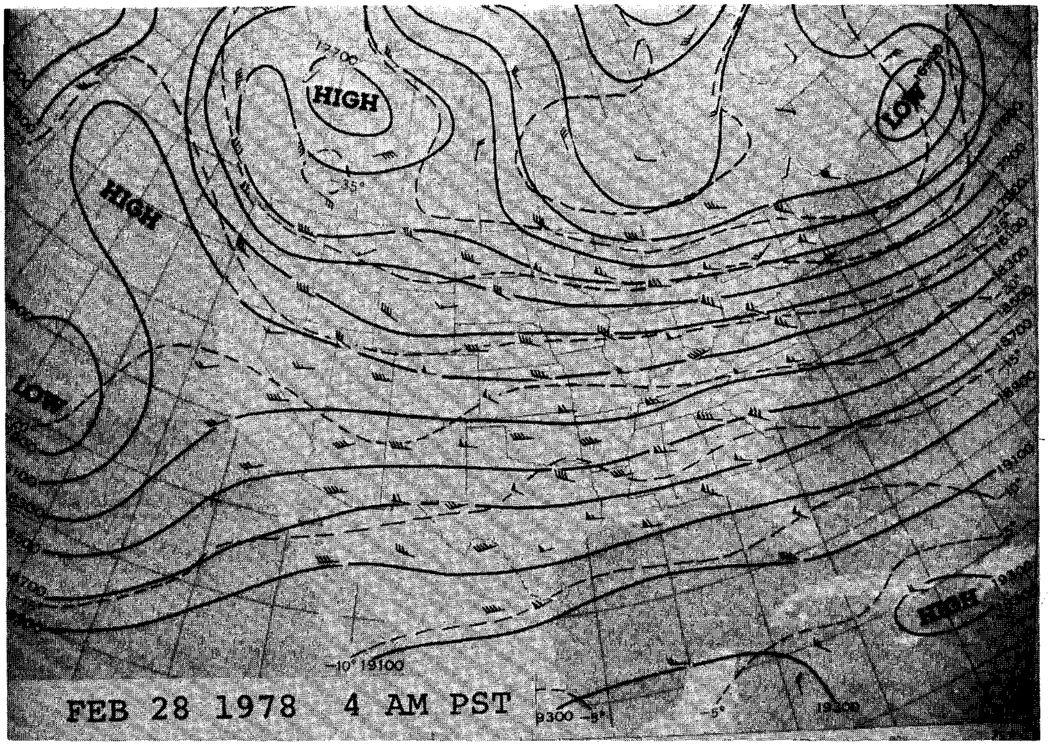


FIGURE 9 500 mb analysis for February 28, 1978.

were the rapid succession of the 500 mb troughs through southern California and the persistence of the Alaskan block.

By March 3 the upper-level block had been displaced northwestward into the Bering Sea and replaced by a low over Alaska. A surface front, which proved to be the last one in the series, moved into southern California on the morning of March 4 (Figure 10) and was accompanied by heavy rain, thunderstorms, and gale force winds. Late that afternoon a ridge built rapidly along 135°W, signaling a return to a fair weather pattern.

WINTER STORMS OF 1979-80

During 1979-80 southern California again experienced a "wet" winter. Heavy rains occurring over the nine-day period February 13-21 caused over $270 million worth of damage and 18 deaths. Over 1,500 homes were destroyed or damaged, and seven of eight southern California counties were declared major disaster areas. Storm totals reached 30 in. at various mountain stations and exceeded 5 in. in the entire southern California coastal region (Figure 11).

Again, as in the latter storms of the 1977-78 winter season, the meteorological situation leading to the series of rainstorms was the result of a block that formed over British Columbia, causing a more southerly storm track. This blocking pattern allowed for a series of six storms to move through southern California during February 13-21. The storms are shown in

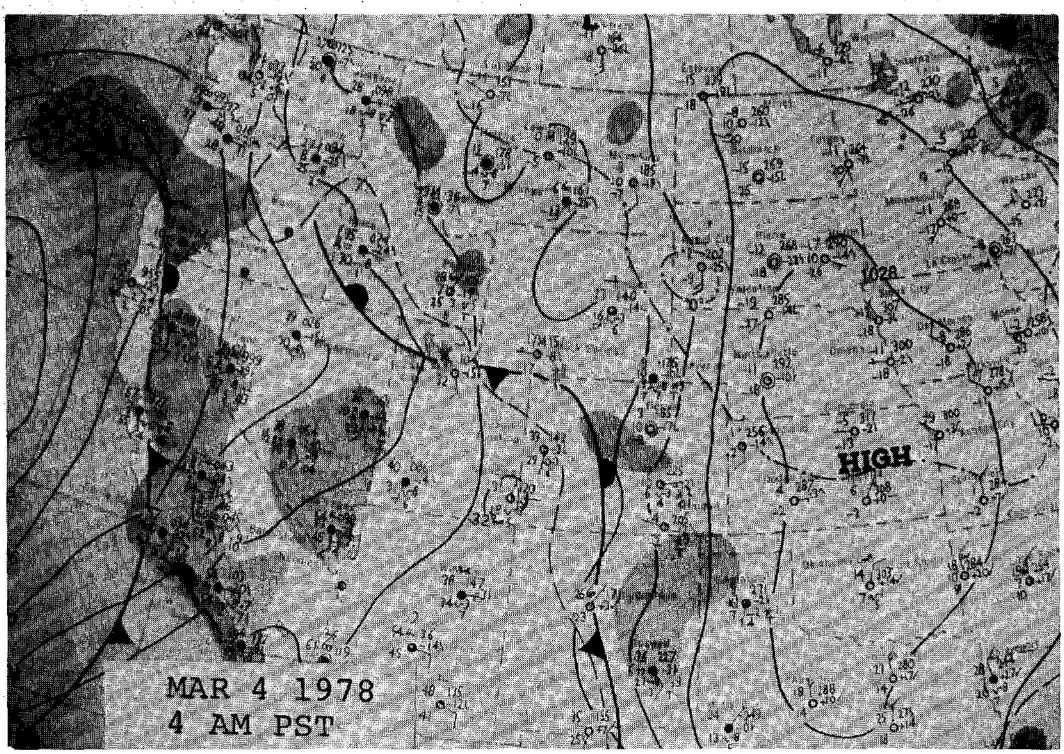


FIGURE 10 Surface weather pattern of March 4, 1978.

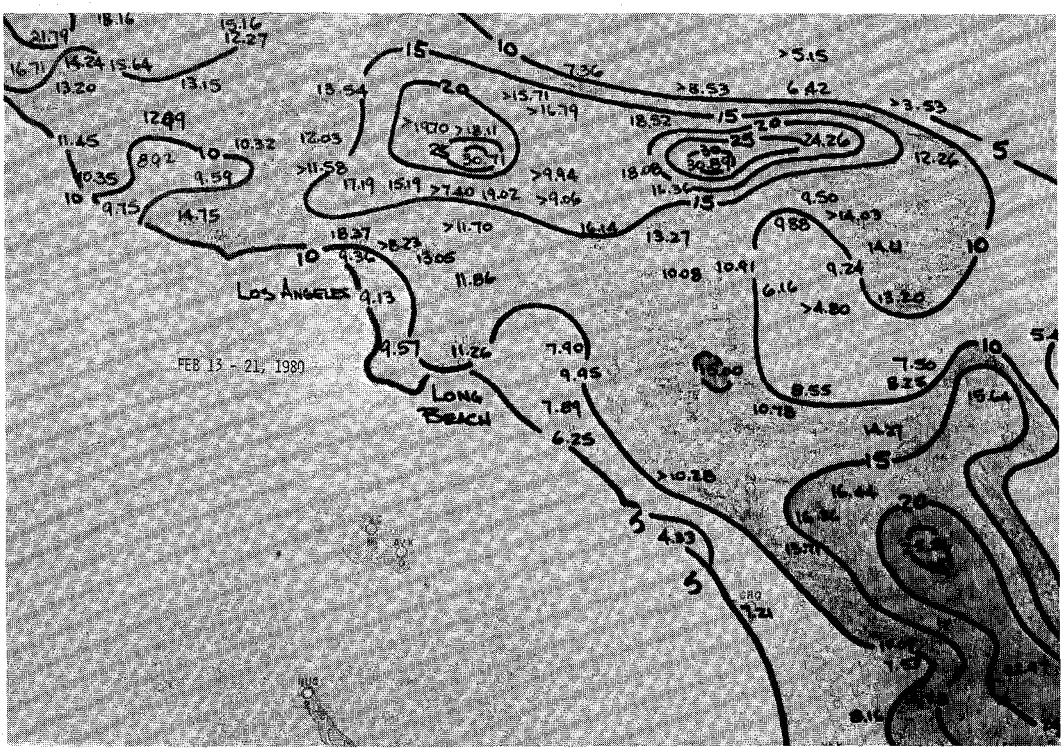


FIGURE 11 Total rainfall from the storms of February 13-21, 1980.

Figures 12-17. The first storm moved inland on the thirteenth (Figure 12) and was followed by a small area of positive vorticity advection (PVA), which was noted just southwest of the main storm.

Storm 2 also began as an area of positive vorticity advection (Figure 13). A frontal band was noted just south of the PVA area, and storm 3 could be seen as a wave on this frontal band, located west of storm 2. The second storm moved inland on the fifteenth, and the proximity of the frontal band to the storm contributed to the heavy rains experienced that day. Twelve hours later storm 3 moved inland (Figure 14), with a strong cyclonic circulation noted northwest of the frontal band. Storm 4 was barely discernible at the western edge of the satellite imagery.

Figure 15 gives a good overview of storms 4, 5, and 6. Storm 4 had just made landfall and was progressing eastward into Arizona. Storm 5, seen in the center of the picture, was intensifying as it moved east at about 30 knots. Storm 6, at the left of the satellite picture, was also forming and moving slowly eastward. As storm 5 moved inland on the twentieth a large area of convective activity intensified just offshore from Los Angeles (Figure 16), testifying to the extremely unstable conditions during that particular day. Storm 6, seen in the extreme left of Figure 16, moved in on the evening of the twentieth and early morning of the twenty-first. However, this storm weakened as it moved onshore due to initial ridging conditions commencing on the twenty-first. Figure 17 shows the ridging effect on the clouds west of southern California. The clouds thinned out as they entered an area of stability.

CONCLUSIONS

The winter storms of 1977-78 and 1979-80 caused extensive damage due to flash floods, floods, and mudslides. The events of these past years and others seem to point out that the potential for widespread destruction is far greater in the wintertime than in any other season. Moreover, winter flooding should not be considered rare events but events that can occur several times within a persons's lifetime. And, as the February 8-10 storm of 1978 proved, devastation can occur even when precipitation amounts fall below 10-year storm totals.

As for the mechanisms that can trigger the extensive and persistent winter rains, there are two distinct types: (1) a well-established high-latitude block and (2) a more southerly storm track that enhances the developmental systems to move through southern California.

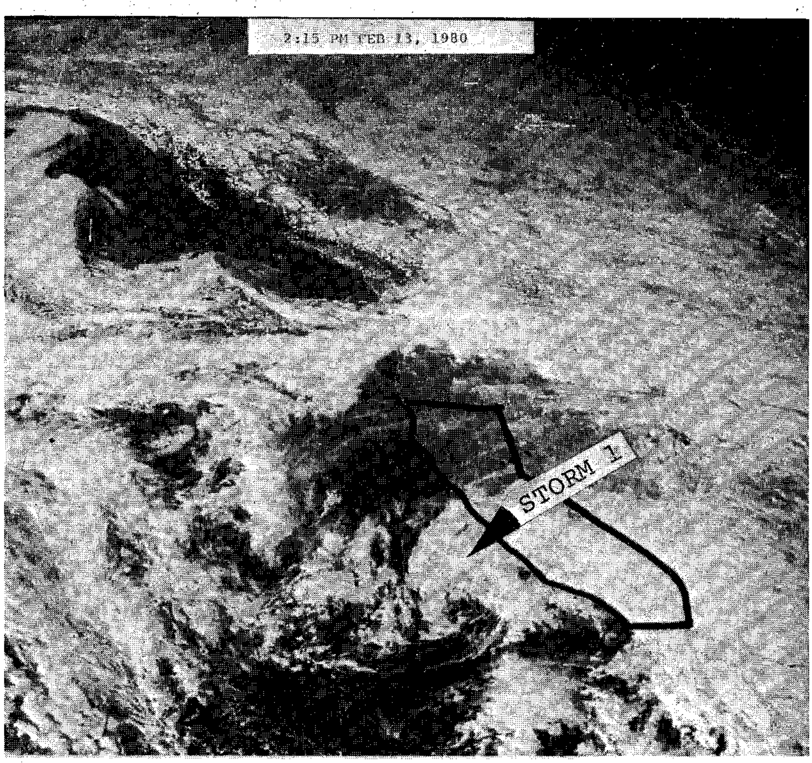


FIGURE 12 Satellite picture of storm 1 on February 13, 1980.

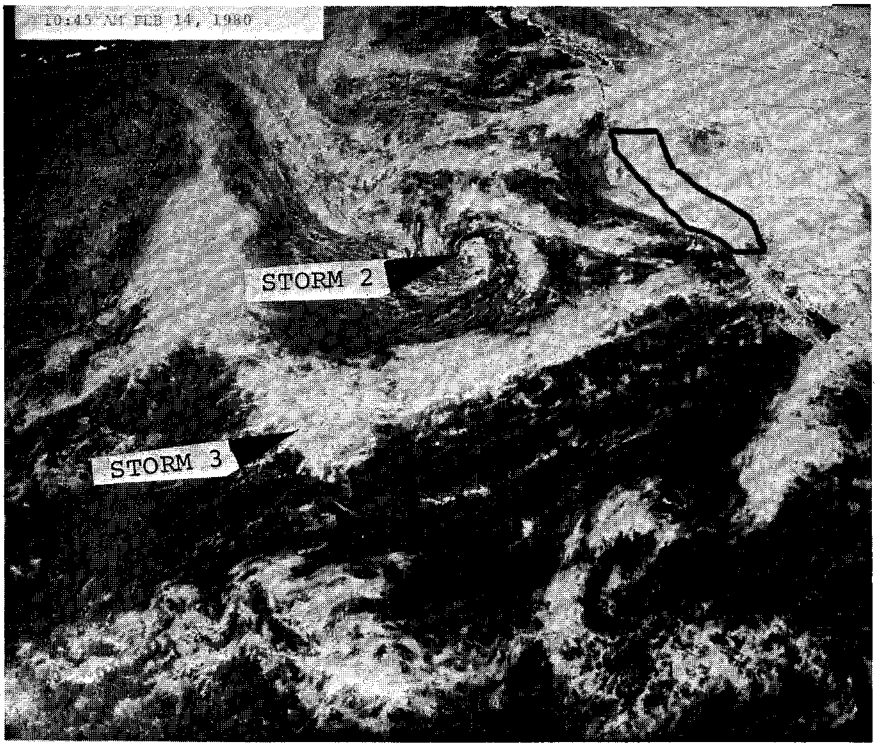


FIGURE 13 Storms 2 and 3 on February 14, 1980, as detected by satellite.

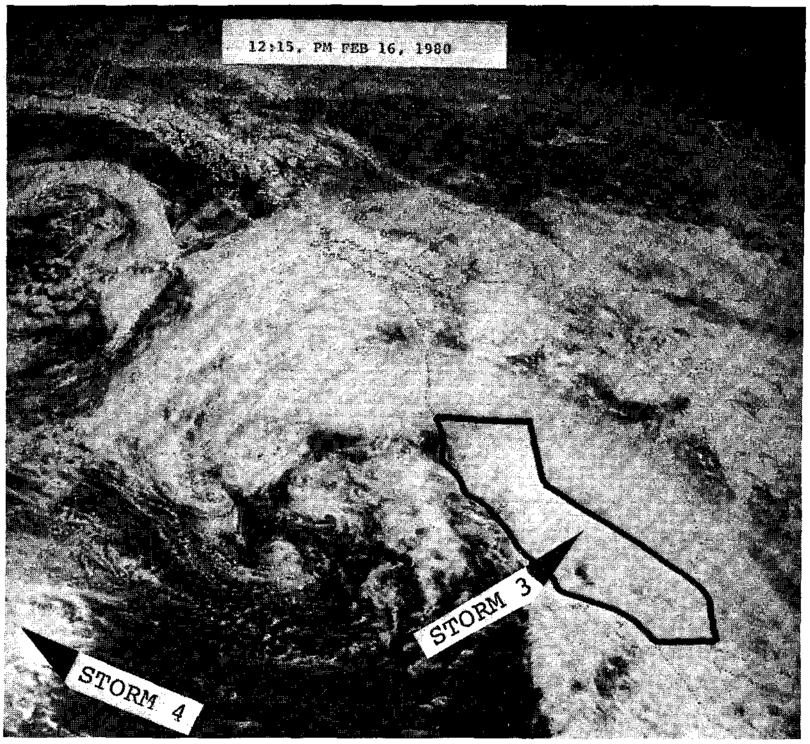


FIGURE 14 Satellite picture of storm 3 moving into Nevada, February 16, 1980.

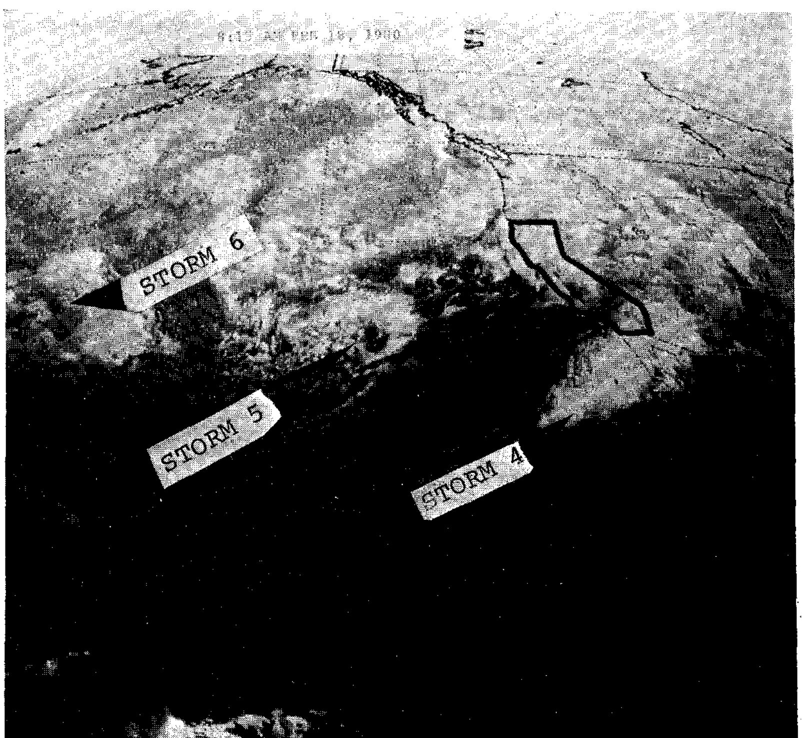


FIGURE 15 Storms 4, 5, and 6 moving eastward on February 17, 1980.

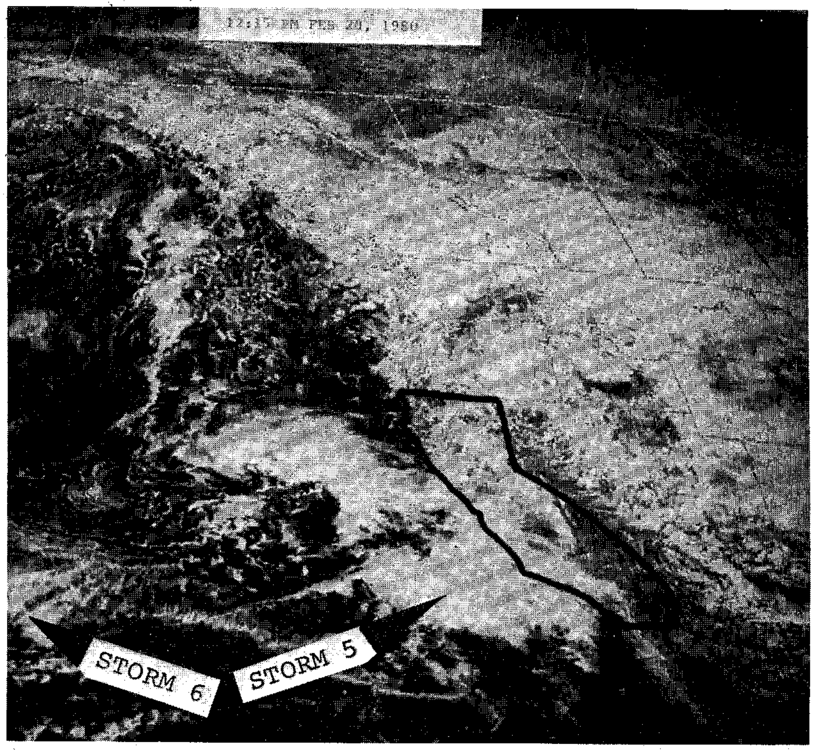


FIGURE 16 Storm 5 approaching Los Angeles on February 20, 1980.

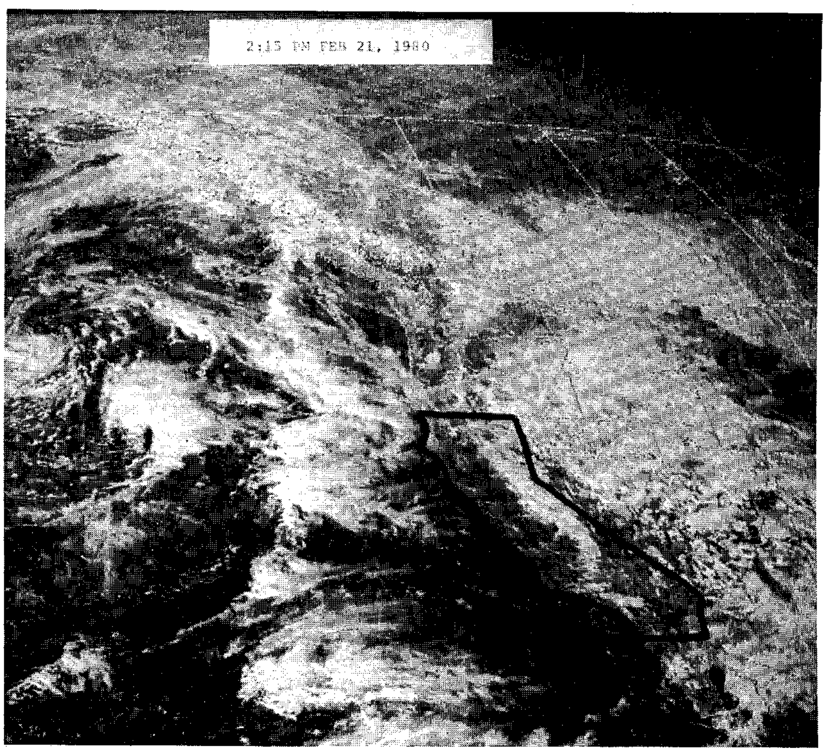


FIGURE 17 Cloud shearing due to building upper-level ridge, February 21, 1980.

HISTORICAL EXTREME ANNUAL RAINFALL DATA IN CALIFORNIA

by James D. Goodridge

All drainage engineering seems to be based around one central idea--the magnitude of the design storm. This paper discusses three data sets that are used in California to compare historical precipitation events and to develop design storms. The first set is an annual series from 740 recording rain gages summarizing 16,000 station-years of data on the extreme annual values of precipitation for 5, 10, 15, 30, and 60 minutes and 2, 3, 6, 12, and 24 hours as well as the annual totals. The second is 36,000 station-years of records from 1,450 nonrecording precipitation gages with extreme annual values for 1, 2, 3, 4, 5, 6, 8, 10, 15, 20, 30, and 60 consecutive days as well as annual totals. The third is the maximum daily precipitation for each month at 1,150 stations with 32,000 station-years of data.

A statistical evaluation of each record is made showing return periods ranging from 2 to 1,000 years. A method of selecting a frequency distribution is illustrated. Regional values of statistical parameters are used in the analysis. This project is a progress report on the joint effort of about 100 public agencies who share data on California's weather. The data sets are updated annually, and the analysis and data are available on microfiche or magnetic tape.

INTRODUCTION

The California Department of Water Resources is involved in assessing extreme rainfall in California and its frequency. These data are used (1) for spillway safety studies on 1,200 dams in the state, (2) for street and roadway drainage studies, and (3) for design of flood control works. These data are extremely useful in comparing current storms with historical events.

Three data files containing extreme rainfall data have been developed to archive these data. Procedures for retrieving and analyzing these data have been developed on a regional basis.

James D. Goodridge is State Climatologist with the California Department of Water Resources in Sacramento, California.

Our data files on extreme annual rainfall data are cooperative efforts of about 100 agencies who keep gages and share their rainfall data.

The 13 regions used to define our extreme annual rainfall data files are shown in Figure 1.

The California Department of Water Resources (1980, 1981) published two reports containing extreme annual precipitation data. The data on depth-duration-frequency of rainfall are divided into two data sets. One set is for recording rain gages. The other is for nonrecording once-a-day observations from the long-term climatological station network.

There are records from 689 recording rain gages representing 15,417 station-years of data. The extreme annual rainfall is indicated for each year for durations of 5, 10, 15, 30, 60, 120, and 180 minutes and for 6, 12, and 24 hours. Return periods indicated are 2 to 10,000 years. These stations are located throughout the state but mainly in the urban areas.

There are records from 853 nonrecording gages representing 31,055 station-years of record for the maximum annual 1, 2, 3, 4, 5, 6, 8, 10, 15, 20, 30, and 60 days as well as for the annual totals. Return periods of 3 to 10,000 years are reported.

Data on the extreme daily rainfall for each month was developed for 1,100 California stations with 32,000 station-years of data. This new report will show extreme rainfall for periods of 2 to 1,000 years.

ANALYSIS

Analysis of extreme precipitation consists of determining four basic statistical parameters: the mean, the standard deviation, the coefficient of skew, and the coefficient of kurtosis. Certain statistical equations basic to the analysis are presented below. These are the equations of mean ($\bar{X}$), standard deviation (S), coefficient of skew (g), coefficient of kurtosis (k), and coefficient of variation (CV):

$$\bar{X} = \frac{\Sigma X}{N}$$

$$S^2 = \frac{\Sigma(x)^2}{N - 1}$$

$$g = \frac{N\Sigma(x)^3}{(N - 1)(N - 2)S^3}$$

$$k = \frac{N^2\Sigma(x)^4}{(N - 1)(N - 2)(N - 3)S^4}$$

$$CV = \frac{S}{\bar{X}}$$

where X = the magnitude of an event, $x = X - \bar{X}$, and N = the number of events.

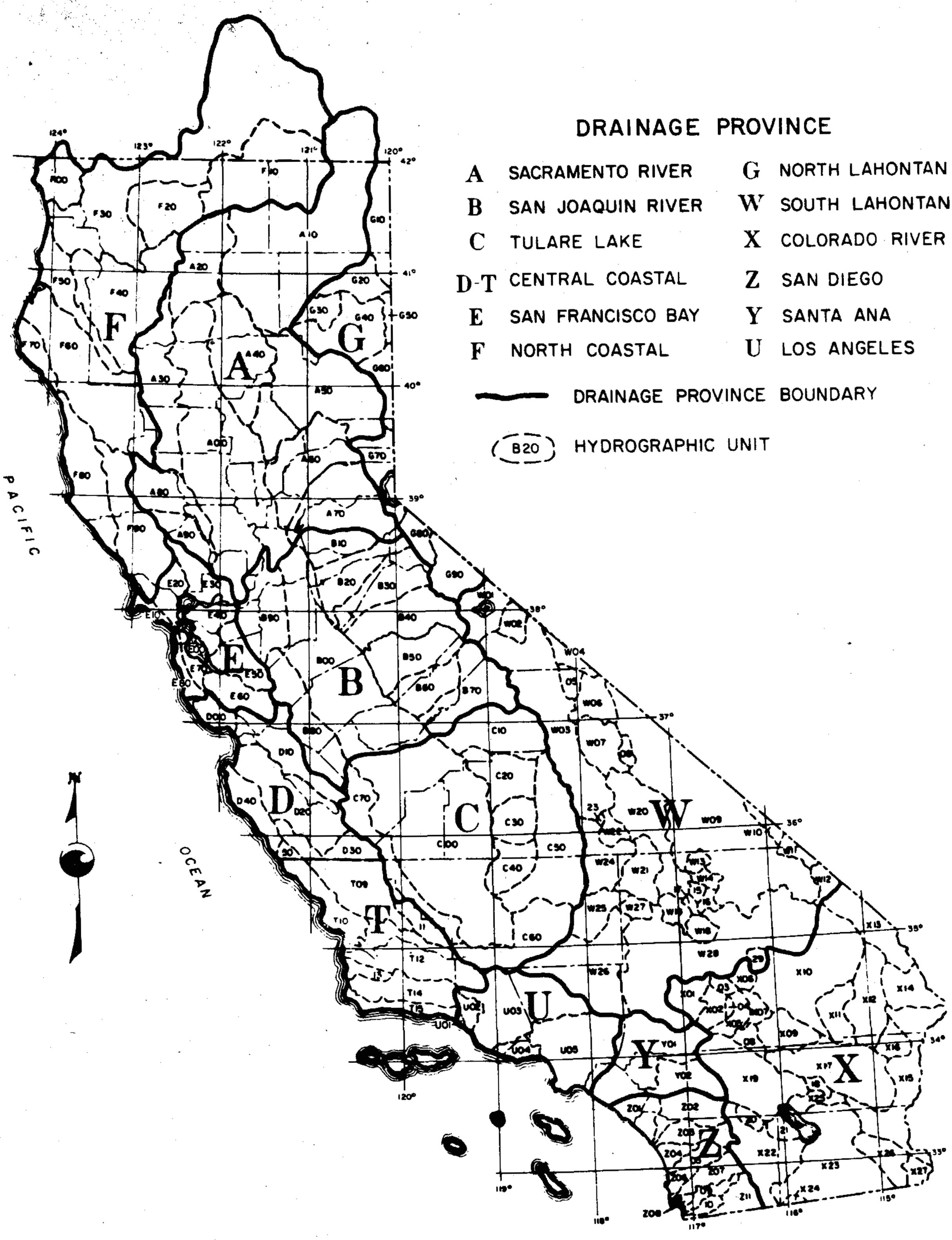


FIGURE 1 Major drainage provinces and hydrographic units.

The mean expresses the central tendency of the data set. The standard deviation measures the deviation from the mean. The skew shows the lack of symmetry, or lopsidedness, that occurs in a rainfall data set due to bounding by a lower limit at one end and by virtually no upper rainfall limit at the other. Kurtosis is used here to test for the appropriateness of various frequency distributions. The coefficient of variation measures the variation of the data set.

RETURN PERIODS

The computed return periods are the mean value plus a varying number of standard deviations. The number of standard deviations corresponding to a return period depends only on the coefficient of skew for the Pearson type III distribution used here. Table 1 shows the general relationship.

TABLE 1 Number of Standard Deviations in Excess of Means Associated with Specific Return Periods

Return Period (years)	Coefficient of Skew					
	0	1	2	3	4	5
2	0	0	-0.31	-0.40	-0.41	-0.38
10	1.28	1.34	1.30	1.18	1.00	0.80
100	2.33	3.02	3.61	4.05	4.37	4.57
1,000	3.09	4.53	5.91	7.15	8.25	9.22

Source: Harter (1967).

The standard deviations in this analysis are expressed as coefficients of variations. The reason for this is the areal stability of coefficient of variation, which is shown in Figure 2. This procedure was reported in the British Flood Studies Report (Sutcliff et al., 1975), as shown in Figure 3.

One of the problems of this type of analysis is the abrupt differences in design values of statistical parameters in adjacent watersheds. An example is the 0.448 value of the coefficient of variation used in the region north of the Mono Lake drainage and the 0.584 value in the Mojave Desert, as shown in Figure 2.

The stability of the coefficient of variation for various storm durations is shown in Figure 4. The monthly variation is illustrated in Figure 5.

The choice of frequency distribution was based on the relationship between the coefficients of skew and kurtosis, as illustrated in Figure 6 for annual series data and in Figure 7 for monthly data. The Pearson type III

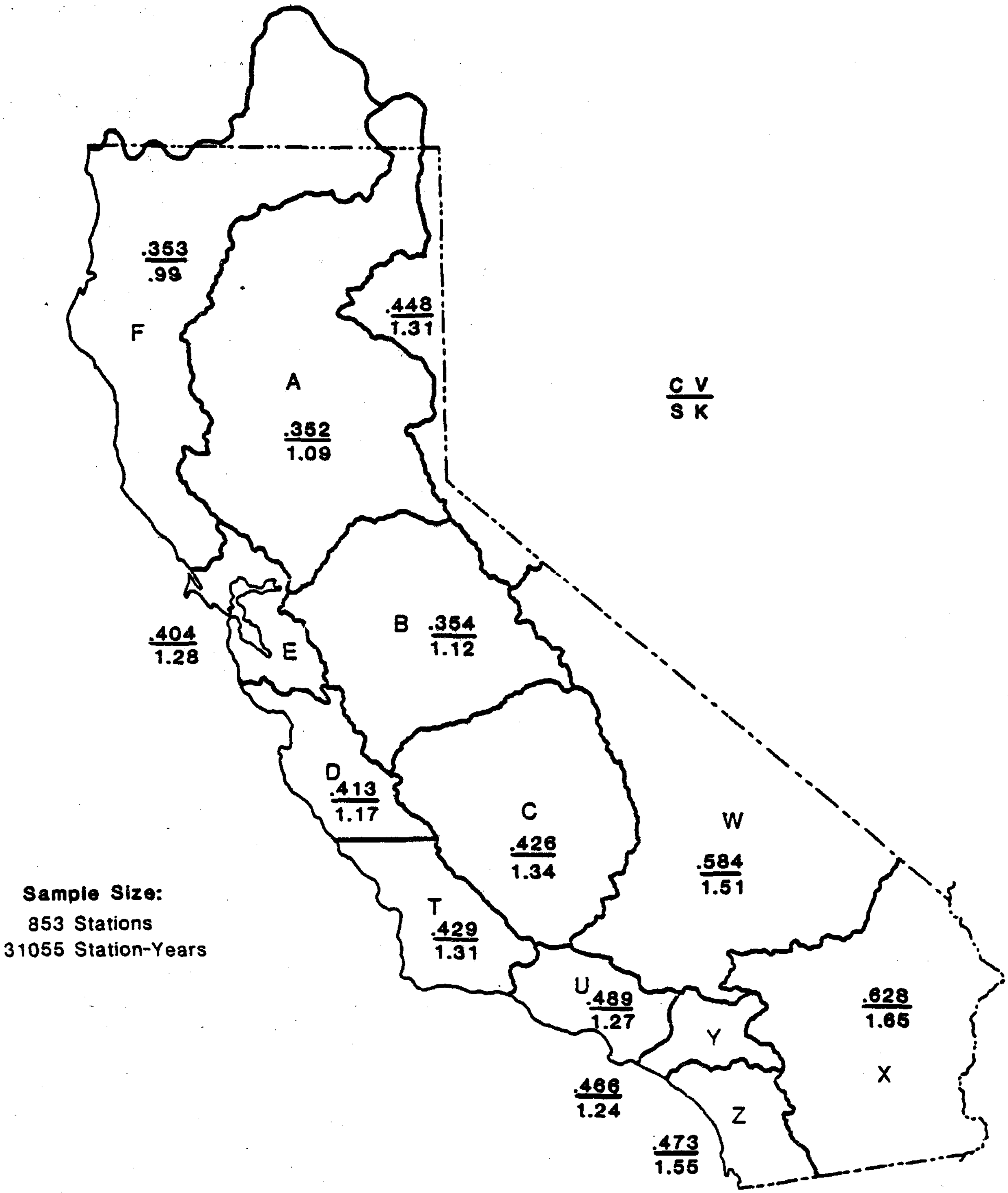


FIGURE 2 Coefficients of variation (above line) and skew (below line) for maximum annual one-day rainfall.

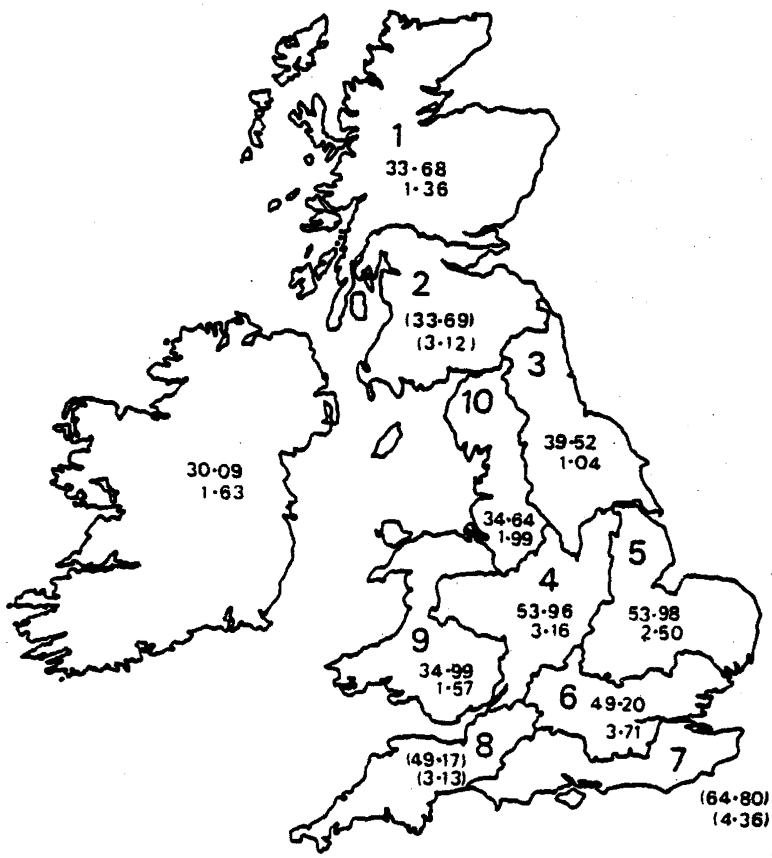


FIGURE 3 An example of another use of regional average values of the coefficients of variation (CV) and skew (SK) (Sutcliff et al., 1975). Upper values = CV x 100; lower values = SK.

distribution was chosen for use in our extreme precipitation studies based on the skew-kurtosis graphs (Wu and Goodridge, 1976). The poor fit of the dry summer months with respect to the Pearson type III criteria is an area needing further study.

EXTREME RAINFALL

The records of extreme annual rainfall for California are shown in Tables 2, 3, and 4. Table 2 shows the extreme rainfall for 5 minutes through 24 hours in each of the 13 major drainage provinces outlined in Figure 1. The greatest 5-minute rainfall total for the state was 1.17 in. at Opid's Camp in the San Gabriel Mountains on April 5, 1926.

The extreme annual rainfall for 1 to 60 consecutive days in each major drainage province is listed in Table 3.

The maximum daily rainfall for each month and each major drainage province is shown in Table 4.

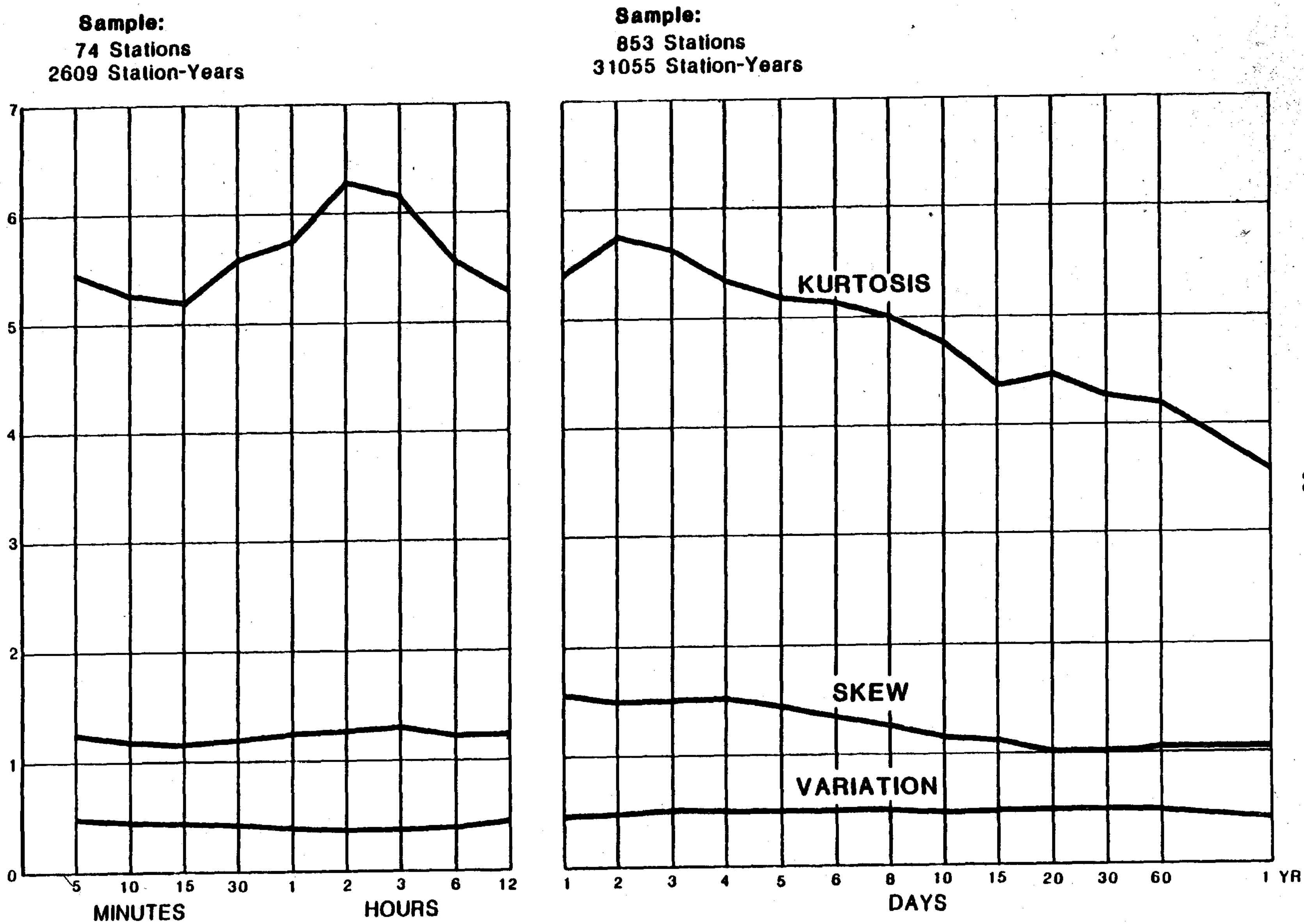


FIGURE 4 Variation in extreme annual precipitation statistics.

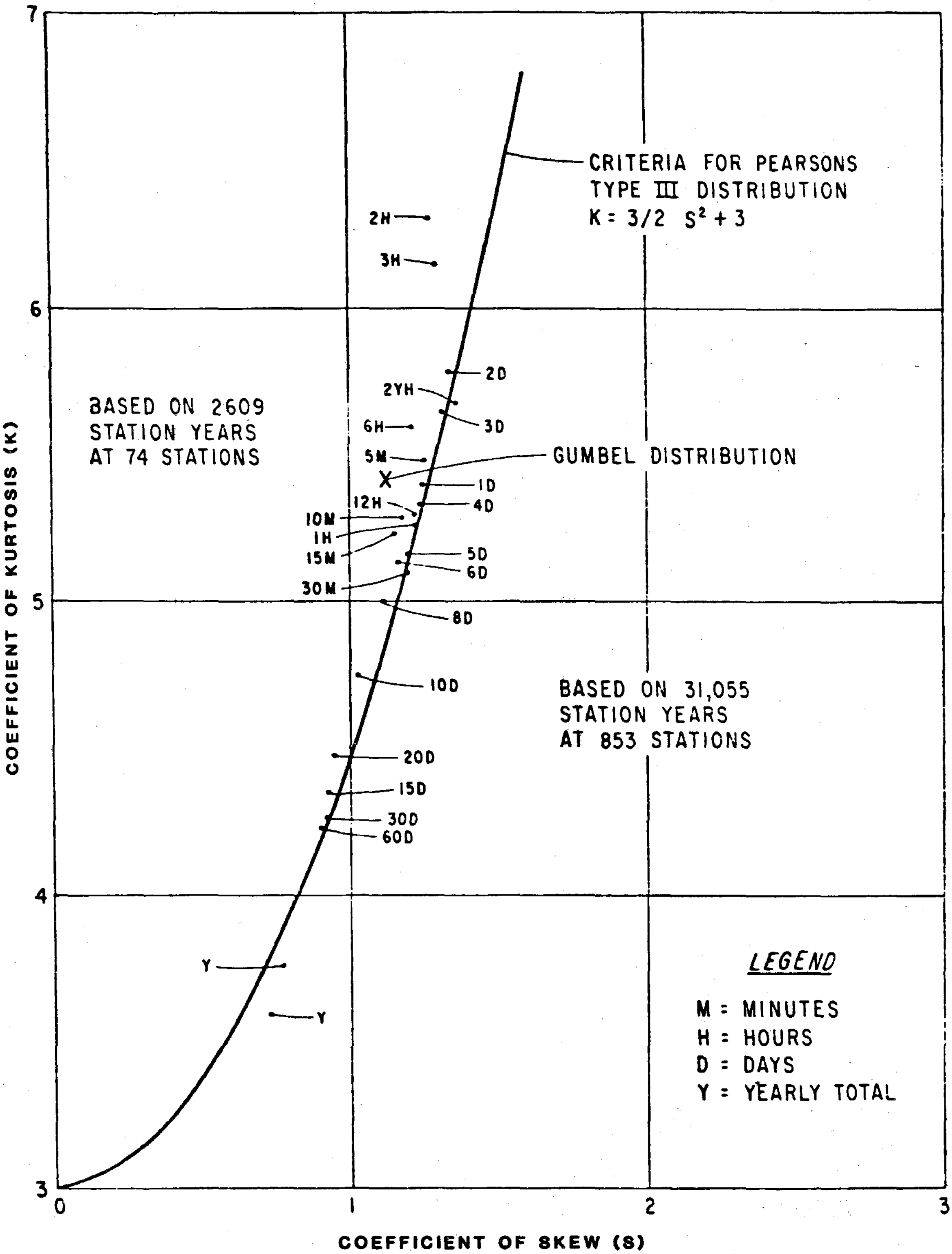


FIGURE 5 Variation in maximum daily rainfall statistics.

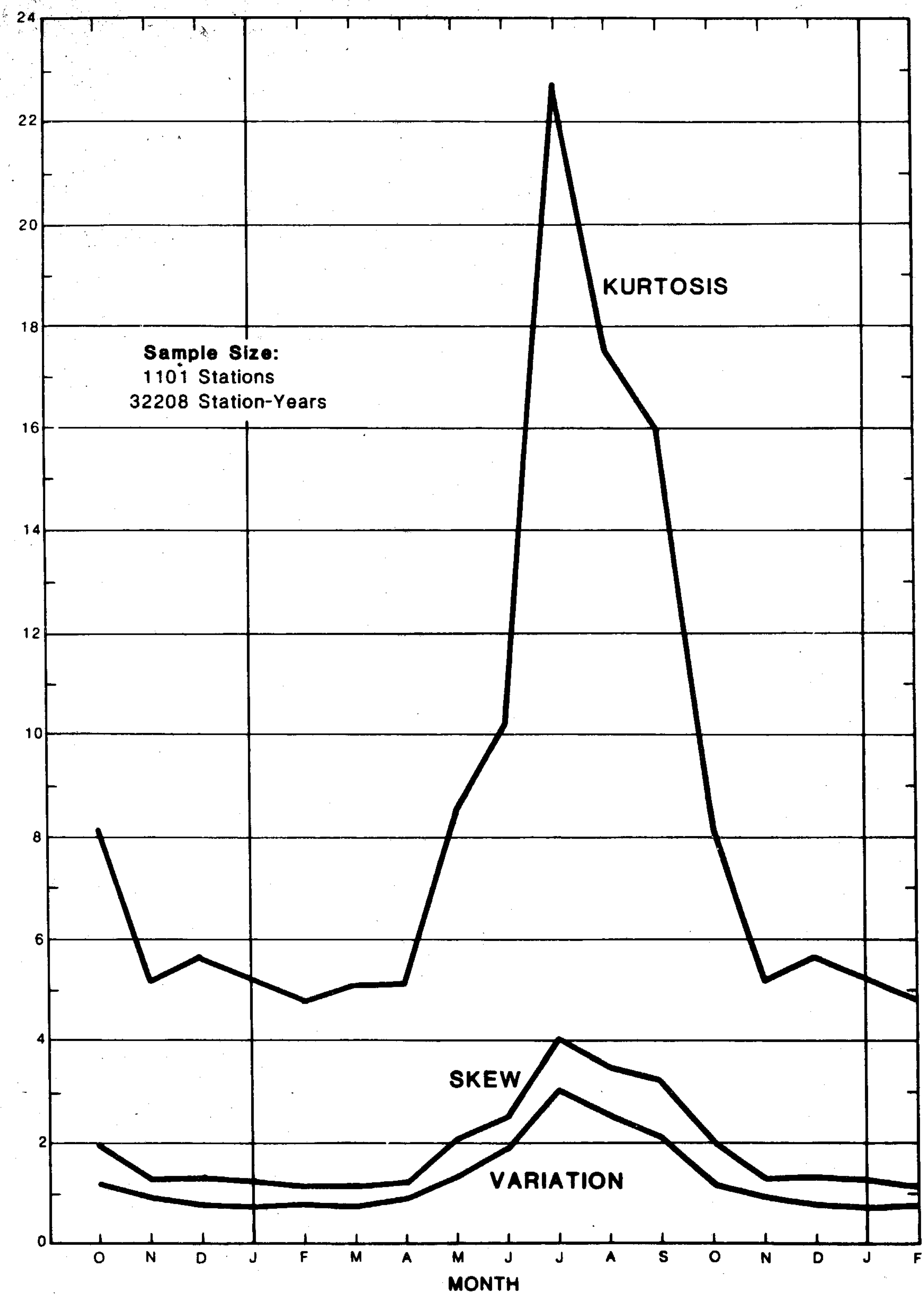


FIGURE 6 Skew-kurtosis relationship of maximum annual precipitation for California.

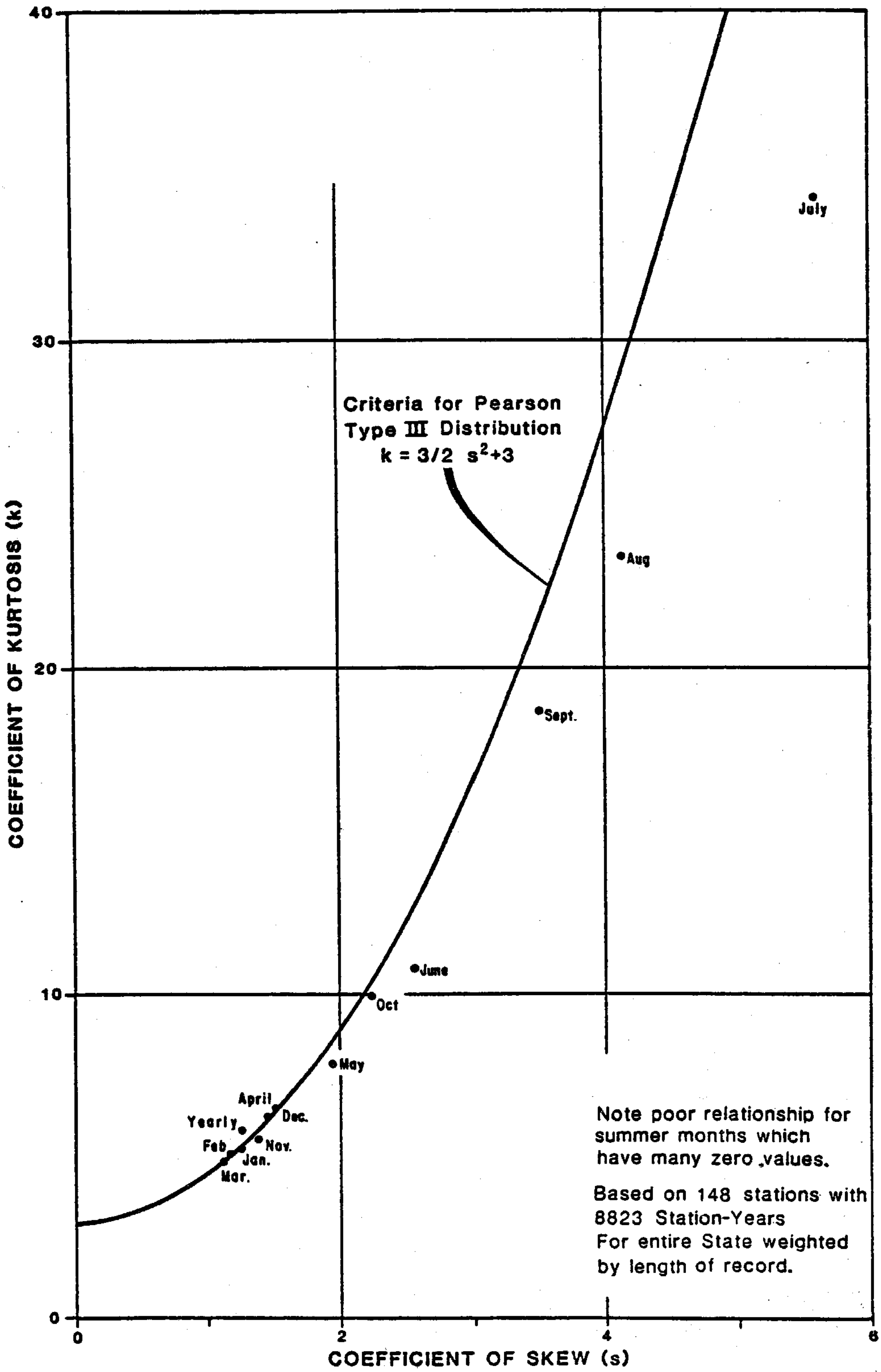


FIGURE 7 Skew-kurtosis relationship of monthly precipitation for California.

TABLE 2 Extreme Annual Short-Duration Rainfall (in.)

Drainage Province	Duration										
	5 min	10 min	15 min	30 min	60 min	2 hr	3 hr	6 hr	12 hr	24 hr	Annual
A	0.87	0.96	1.44	1.94	2.46	3.73	4.53	5.51	8.90	12.97	123.59
B	0.57	1.05	1.57	2.00	2.33	2.36	3.33	4.08	6.11	9.41	81.51
C	0.65	1.30	1.52	1.62	2.13	3.90	4.70	6.41	8.55	13.58	80.22
D	0.63	0.77	1.00	1.31	1.75	2.56	3.35	5.87	9.62	14.09	98.21
E	0.50	0.74	1.40	1.40	1.90	3.65	5.25	8.42	10.19	11.11	71.56
F	0.75	0.85	1.04	1.50	2.25	2.75	3.40	4.70	8.60	13.21	120.10
G	0.61	0.90	1.15	1.35	1.49	1.63	1.87	2.21	3.51	5.95	73.71
T	0.85	0.96	1.25	1.55	2.05	3.09	4.37	7.86	10.65	14.24	81.17
U	1.17	1.18	1.40	2.31	2.70	3.83	5.20	7.24	13.38	26.12	89.07
W	1.00	1.14	1.42	1.70	2.01	2.67	3.71	5.90	9.80	14.50	96.10
X	0.62	0.85	0.91	1.45	2.30	2.90	2.56	3.34	5.75	8.28	43.62
Y	0.70	1.21	1.44	1.72	2.10	2.62	3.70	7.39	10.10	17.25	75.21
Z	0.60	1.05	1.48	2.17	2.67	2.75	2.90	4.70	7.24	9.83	71.30
State	1.17	1.30	1.57	2.31	2.70	3.90	5.25	8.42	13.38	26.12	123.59

Note: Sample size was 655 stations for 14,632 station-years.

TABLE 3 Extreme Annual Long-Duration Rainfall (in.)

Drainage Province	Duration Days 1	2	3	4	5	6	8	10	15	20	30	60	Annual
A	14.20	21.53	25.28	27.93	31.80	34.84	36.57	40.05	45.34	57.10	57.53	80.84	129.24
B	9.04	15.95	19.48	19.83	20.40	24.20	25.58	27.97	29.79	38.25	40.20	50.03	80.91
C	14.94	21.40	27.30	27.31	30.45	30.46	33.32	34.46	37.12	39.41	42.39	62.77	87.45
D	12.57	16.79	20.18	22.02	26.38	26.84	27.69	29.85	33.30	35.53	45.24	60.50	100.16
E	12.01	17.75	21.56	24.49	33.86	39.11	42.27	43.20	44.85	46.87	51.81	70.00	89.88
F	13.65	22.73	25.62	31.90	37.60	40.10	49.20	52.00	54.10	61.00	81.45	90.35	174.40
G	8.76	12.70	15.55	17.40	18.99	21.03	22.50	24.93	27.43	31.79	33.79	43.07	60.88
T	16.00	21.00	22.02	22.70	28.63	33.63	43.41	43.81	45.04	50.41	51.77	72.64	80.13
U	20.57	36.34	36.94	37.34	37.53	38.20	44.97	49.01	54.86	56.64	57.42	79.65	91.79
W	16.81	22.96	25.66	26.87	29.38	34.52	41.10	44.41	45.54	84.72	88.03	85.64	98.24
X	13.50	16.30	18.85	22.85	23.70	24.20	24.24	26.90	29.57	29.57	31.30	41.01	74.54
Y	24.92	32.10	33.40	33.80	36.58	43.15	49.10	50.85	53.15	53.70	56.70	88.50	99.70
Z	14.48	22.40	23.40	25.44	26.35	26.93	31.50	32.50	32.50	36.05	38.89	51.99	73.17
State	24.92	36.34	36.94	37.34	37.60	43.15	49.20	52.00	54.86	84.72	88.03	90.35	174.40

Note: Sample size was 853 stations for 31,055 station-years.

TABLE 4 Maximum Daily Rainfall for California (in.)

Drainage Province	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Year
A	11.40	8.56	15.34	11.75	10.92	9.16	5.60	5.68	5.48	5.96	3.90	8.58	15.34
B	4.97	9.29	9.72	10.80	10.07	7.50	5.70	4.57	2.11	3.14	2.59	5.57	10.80
C	3.70	9.55	14.94	9.33	9.95	6.21	4.89	6.32	3.04	1.13	2.03	5.25	14.94
D	11.15	11.91	10.42	10.40	10.30	6.46	5.75	4.47	1.88	2.51	1.42	8.35	11.91
E	13.79	9.31	11.09	11.56	8.35	8.35	4.95	3.95	2.65	2.45	1.92	6.09	13.79
F	11.50	12.40	11.30	9.90	9.78	8.90	6.90	5.97	4.87	4.74	3.65	7.35	12.40
G	4.85	4.66	7.60	6.78	8.76	3.60	4.15	2.48	4.50	3.51	2.78	2.60	8.76
T	2.97	6.52	8.14	16.00	10.06	6.85	4.88	3.22	1.73	1.30	1.50	3.55	16.00
U	6.75	9.69	13.66	20.57	20.00	14.92	6.55	4.50	1.34	1.07	3.48	9.02	20.57
W	7.35	9.94	11.14	15.86	14.00	12.62	5.15	4.04	2.12	2.50	3.70	8.00	15.86
X	5.00	9.50	8.00	11.50	7.60	13.50	5.60	2.01	1.12	3.25	4.01	6.45	13.50
Y	6.37	12.40	15.15	21.61	9.73	15.06	5.22	3.63	1.63	3.90	3.80	8.13	21.61
Z	5.52	9.60	7.92	8.54	12.81	7.65	5.33	3.69	2.25	2.87	3.67	5.00	12.81
State	13.79	12.40	15.34	21.61	20.00	15.06	6.90	6.32	5.48	5.96	4.00	9.02	21.61

Note: Sample size was 1100 stations for 32,200 station-years.

The heaviest 24-hour rainfall in California was the 26.12 in. recorded on January 22, 1943, at Hoegees in the San Gabriel Mountains of Los Angeles County at 2,650 ft. This station, which receives an average annual rainfall of 37 in., receives 64 percent of the annual total gage catch during the 60 wettest days of the year. Maximum 24-hour precipitation for each drainage province and each month is shown in Figure 8.

Extreme rains do not necessarily fall just where rain gages are located, just as some recorded rainfall extremes are for records that are not included in our summaries of extreme rainfall. The 8 in. of rain in 2.5 hours measured in Chiatovich Flat in the White Mountains by a traveler (Douglas Powell) who was carrying a rain gage illustrates this point. Some of these extreme events are shown in Figure 9.

OUTLIERS

The effect of extreme storms on the coefficients of skew and variation is minimized by areal averaging. Extreme storms, called "outliers," do not seem to fit with the data sets in which they are found. An example is the September 24, 1939, storm at Indio in Riverside County, when 158 mm (6.45 in.) fell in six hours (Pyke, 1975). Yet the mean annual rainfall at Indio is only 19.25 mm (3.12 in.).

Another example was the 140 mm (5.5 in.) that fell in Citrus Heights on October 12, 1962. The coefficient of variation of the maximum annual one-day rainfall of the 23-year record at Citrus Heights is 0.55 without the extreme storm of 1962, but 0.90 when it is included.

The outliers have been included in the records contained in this paper so that their influence on the development of design storms will be felt. In the past some analysts excluded outliers as a means of providing less costly drainage facilities.

DESIGN STORMS

Selecting a design storm using the procedures outlined here consists of using the mean storm plus a varying number of standard deviations in excess of the mean. A map of the once-in-a-1,000-year 24-hour rainfall for California is shown in Figure 10 as a percentage of the mean annual rainfall. This once-in-a-1,000-year storm varies from 200 percent of the mean annual rainfall in the desert area near the Salton Sea to less than 20 percent of the mean annual precipitation in the more humid areas of the state.

STATION-YEAR METHOD

The procedure used here for averaging coefficients of variation, skew, and kurtosis is essentially a variation of a station-year method. There was an extensive discussion of this method in a paper by Katherine Clark-Hafstad (1942). Estimates of frequencies of rare events by a station-year approach have a weakness in that there is some interdependence in the data from adjacent stations. Studies of the deteriorating determination coefficients of

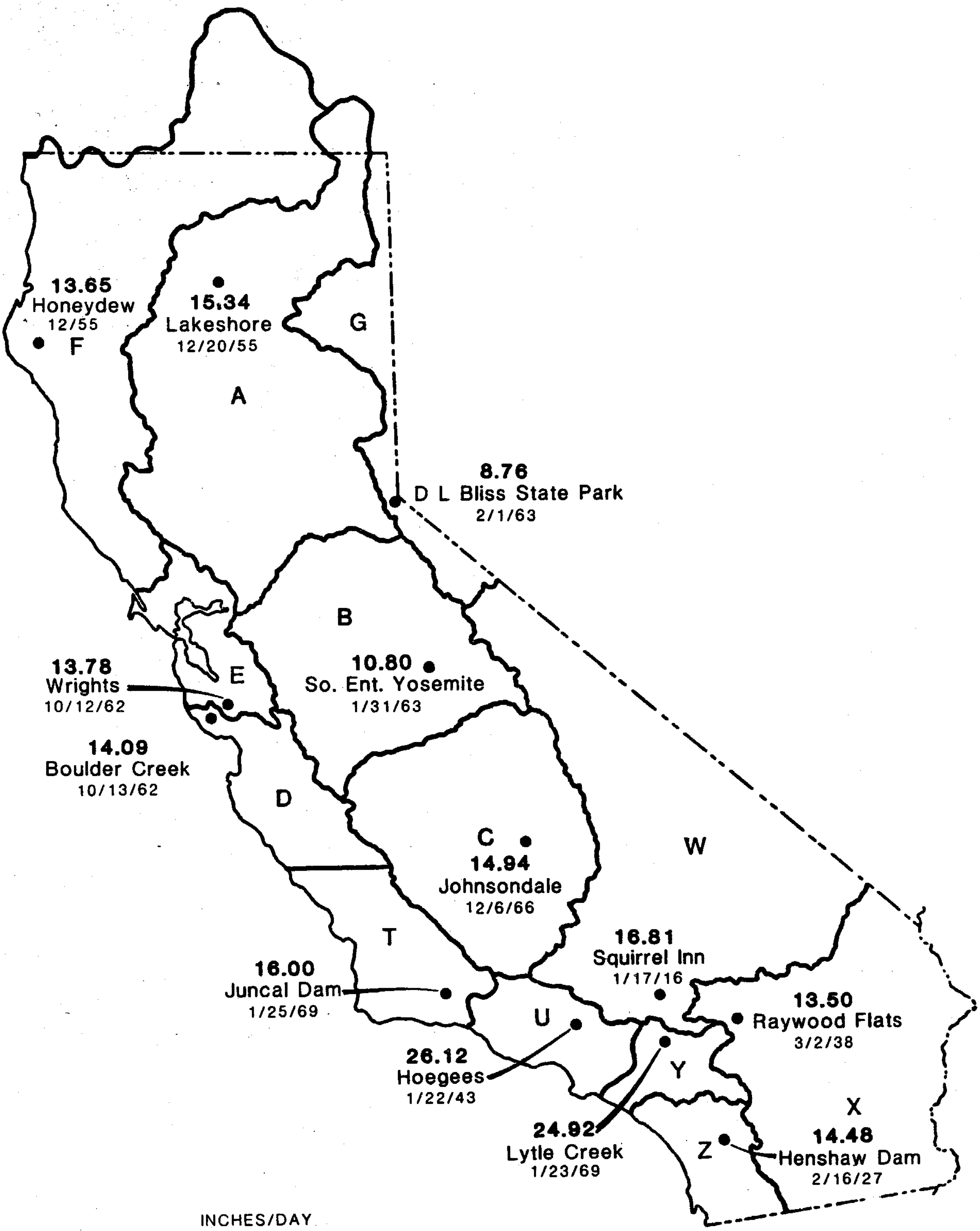
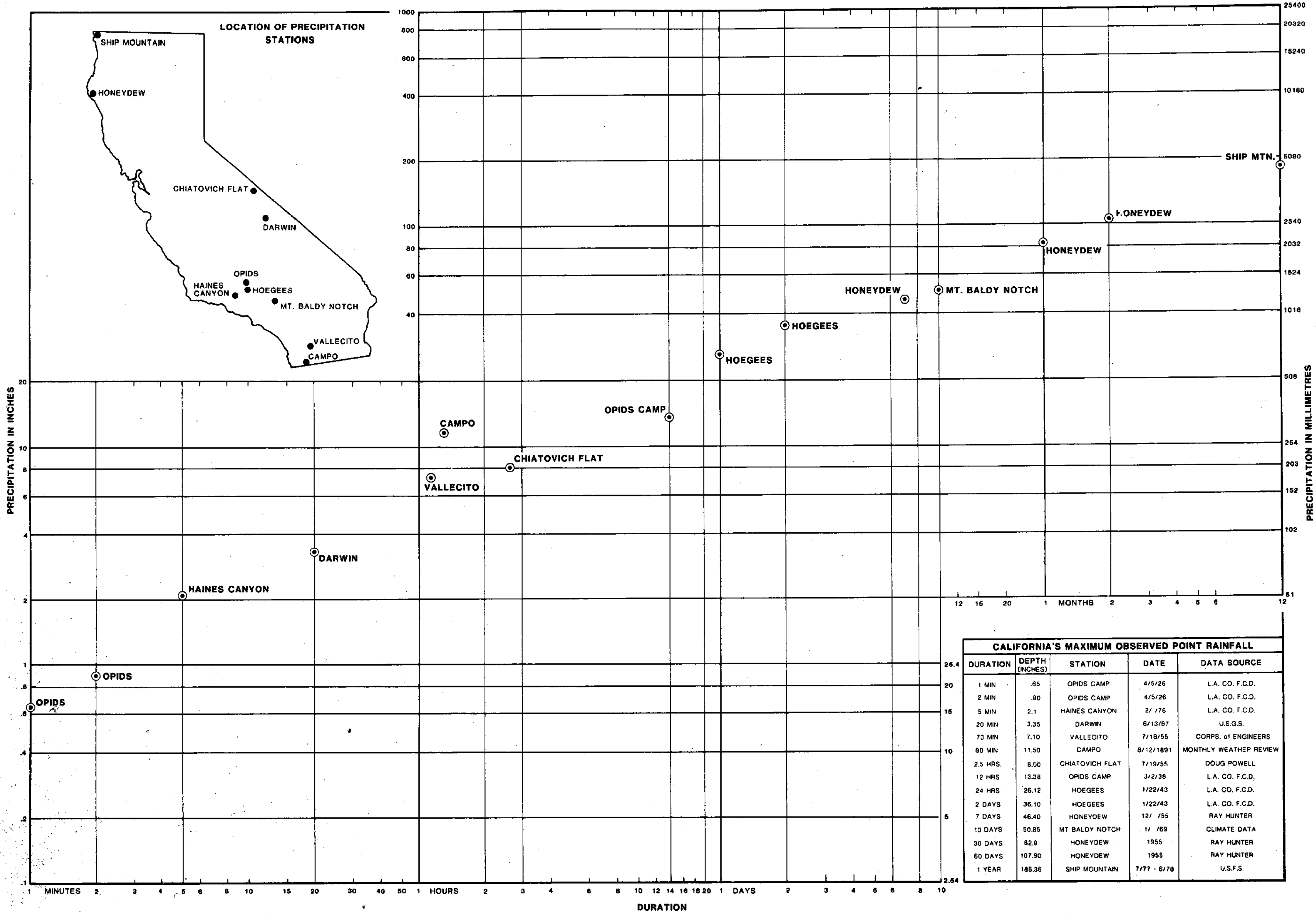


FIGURE 8 Maximum daily rainfall.



CALIFORNIA'S MAXIMUM OBSERVED POINT RAINFALL

DURATION	DEPTH (INCHES)	STATION	DATE	DATA SOURCE
1 MIN	.65	OPIDS CAMP	4/5/26	L.A. CO. F.C.D.
2 MIN	.90	OPIDS CAMP	4/5/26	L.A. CO. F.C.D.
5 MIN	2.1	HAINES CANYON	2/ /76	L.A. CO. F.C.D.
20 MIN	3.35	DARWIN	6/13/67	U.S.G.S.
70 MIN	7.10	VALLECITO	7/18/55	CORPS. of ENGINEERS
80 MIN	11.50	CAMPO	8/12/1891	MONTHLY WEATHER REVIEW
2.5 HRS.	8.00	CHIATOVICH FLAT	7/19/55	DOUG POWELL
12 HRS	13.38	OPIDS CAMP	3/2/38	L.A. CO. F.C.D.
24 HRS	26.12	HOEGEES	1/22/43	L.A. CO. F.C.D.
2 DAYS	36.10	HOEGEES	1/22/43	L.A. CO. F.C.D.
7 DAYS	46.40	HONEYDEW	12/ /55	RAY HUNTER
10 DAYS	50.85	MT BALDY NOTCH	1/ /69	CLIMATE DATA
30 DAYS	82.9	HONEYDEW	1955	RAY HUNTER
60 DAYS	107.90	HONEYDEW	1955	RAY HUNTER
1 YEAR	185.36	SHIP MOUNTAIN	7/77 - 6/78	U.S.F.S.

FIGURE 9 Depth-duration relationship of California's greatest observed rainfalls.

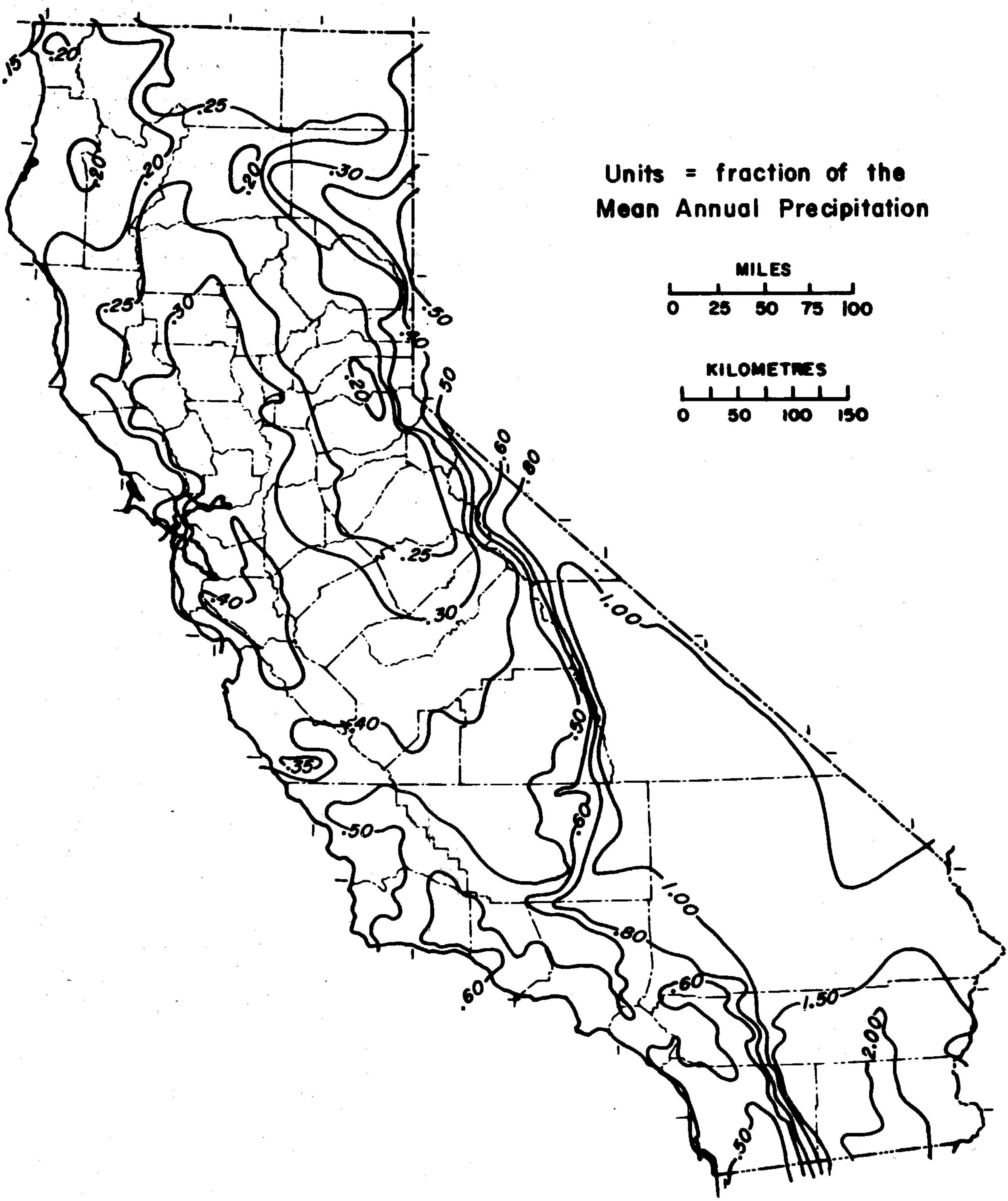


FIGURE 10 Once-in-a-1,000-year 24-hour storms.

rainfall from extreme storms have shown that the effective amount of data can be estimated. These studies were not done for this report because all of the available data have been used in the development of our regional coefficients.

The main object of this paper is to describe the data sets and the extreme values. These data are readily available to those who need their own analysis.

REFERENCES

California Department of Water Resources (1980) Maximum Daily Precipitation by Months, California Department of Water Resources, Sacramento.

California Department of Water Resources (1981) Rainfall Depth-Duration-Frequency for California, California Department of Water Resources, Sacramento.

Clark-Hafstad, Katherine (1942) Reliability of station-year rainfall-frequency determinations, Transactions of the American Society of Civil Engineers 107:633-683.

Harter, H. Leon (1967) Central Technical Unit, U.S. Department of Agriculture Soil Conservation Service, Hyattsville, Md.

Jennings, Arthur H. (1952) Maximum 24-Hour Precipitation in the United States, Technical Paper 16, U.S. Weather Bureau, Washington, D.C.

Pyke, Charles B. (1975) Indio, California, thunderstorm of 24 September 1939, Precipitation Analysis for Hydrologic Modeling, Proceedings of a national symposium sponsored by the Precipitation Committee, Section on Hydrology, American Geophysical Union, June 26-28, 1975, Davis, California.

Sutcliff, J. V., et al. (1975) Flood Studies Report, Natural Environmental Research Council, 27 Charing Cross Road, London.

U.S. Weather Bureau (1959) Maximum Station Precipitation for 1, 2, 3, 6, 12, and 24 Hours, Part XXIII California, Technical Paper 15, U.S. Weather Bureau, Washington, D.C.

Wu, Baolin, and Goodridge, J. D. (1976) Selection of Frequency Distribution, With Tables for Pearson Type III, Log-Normal Weibull, Normal, and Gumble Distributions, California Department of Water Resources, Sacramento.

APPENDIX: BIBLIOGRAPHY OF PUBLICATIONS ON CLIMATOLOGY

Winds Along the Los Angeles Aqueduct, Warren W. Rodie, California State University, Northridge, November 1978.

Effects of Tropical Cyclones Upon Southern California, Michael Ford Harris, California State University, Northridge, June 1969.

Estimation of the Annual Potential Wind Power Budget in a High Mountain Region from Historic Synoptic Weather Data, David E. Schorran, University of Nevada, Reno, April 1978.

The History of Temperature Observations in the City of San Francisco--1850-1880 and an Introduction to the Seasonal and Annual Variations of Temperature--1850-1974, Tom Loffman, Berkeley, June 1975.

Climatic Summaries for Military Weather Stations, January 1978.

Climatic Summaries of California Airports, August 1978.

Guide to Standard Weather Summaries and Climatic Services, prepared by the U.S. Navy at the National Climatic Center, Asheville, North Carolina, January 1978.

Lake Tahoe Water Balance, Rick A. Lind, California State University, Sacramento, June 1978.

Climate of California, C. Robert Elford, reissued June 1970.

Climate of California, National Climatic Center, Asheville, North Carolina, August 1977.

Selection of Frequency Distributions, June 1976.

California Rainfall, monthly total precipitation for each water year starting with 1969, issued each year.

Wind Power on San Nicolas Island, California, Arnold Court, California State University, Northridge, August 1979.

Climatic Summary of the United States, supplement for 1951 through 1960, U.S. Department of Commerce, 1964.

Department of Water Resources Publications

Climatological Stations in California, 1971, Indexed by County, Bulletin 165, July 1971, 114 pp., $2.50.

Evaporation from Water Surfaces in California, Bulletin 73-79, November 1979, $5.

California Rainfall Summary, Monthly Total Precipitation 1849-1979, 55 pp. plus 10 (48x) microfiche, June 1980, $3.

Rainfall Depth-Duration-Frequency for California, 38 pp. plus 21 microfiche, December 1980, $3.

Maximum Daily Precipitation by Months, 36 pp. plus 8 microfiche, October 1980, $3.

Runoff Depth-Duration-Frequency in Selected California Watersheds, memorandum report, January 1973, 125 pp.

Simplified Watershed Analysis--Water Loss Depth-Duration-Frequency on Selected Watersheds in California, November 1973, 34 pp.

California Sunshine--Solar Radiation Data, Bulletin 187, August 1978, 110 pp.

Wind in California, Bulletin 185, January 1978, 267 pp., $3.

Wind Storms in California, December 1979, 34 pp.

Copies of most of the publications listed in this bibliography are available by writing to the State of California, Department of Water Resources, P.O. Box 388, Sacramento, California 95802. For publications for which a charge is made, checks or money orders should be made payable to the State of California. Residents of California should add the 6 percent state sales tax.

RETURN PERIODS OF 1977-80 PRECIPITATION IN SOUTHERN CALIFORNIA AND ARIZONA

by Charles B. Pyke

During the 1977-78, 1978-79, and 1979-80 winter rainfall seasons in southern California and Arizona there were a number of rare precipitation amounts on time scales ranging from a few minutes to an entire season. Between mid-December 1977 and mid-March 1978 most southern California and Arizona stations received 250 to 400 percent of their normal precipitation. The return periods for the totals observed over durations from 70 to 100 days exceeded 20 years at many stations and ranged as high as 165 years. Short-duration high-intensity rainfall at several southern California stations in early 1978 also exceeded 100-year return periods. In January 1978 rainfall amounts of 1.2 in. or more in 15 minutes were observed at widely separated southern California stations, with return periods running as high as 3,300 years. In 1978-79 there were a number of relatively rare three-month rainfall totals in Arizona, but relatively few highly unusual short-duration high-intensity events were observed. In 1979-80 most of the unusual precipitation statistics centered around the storm period of February 13-22, during which many stations exhibited 10-day totals approaching, and in a few cases exceeding, their normal annual precipitation, with return periods exceeding 200 years in some instances. Two high-intensity storms in 1980 also exhibited return periods exceeding 100 years at some stations for durations of four to eight hours.

INTRODUCTION

During the years 1977-80 there were a substantial number of unusually heavy precipitation amounts at various southern California and Arizona stations on time scales ranging from a few minutes to an entire season. Most of these occurred during the winter months, primarily December through March, but a few of the shorter-duration events were observed during the summer and fall.

Following two years of severe drought from 1975 through much of 1977, the winter rainfall seasons of 1977-80 were all considerably wetter than normal in

---

Charles B. Pyke is Meteorologist with the U.S. Army Corps of Engineers in Los Angeles, California.

the southwestern United States, with several especially wet periods. Between mid-December 1977 and mid-March 1978 most southern California and Arizona stations received from 250 to 400 percent of their normal precipitation. Several very heavy storms also occurred in December 1978 and January 1979, especially in Arizona. Still another unusually wet period occurred in southern California and Arizona from early January through early March 1980, with a strong concentration of very heavy storms from February 13 through 22.

LIST OF PRECIPITATION EVENTS

A listing of many of the major heavy precipitation events that occurred in southern California and Arizona from 1976 through 1980 can be found in Table 1. This listing includes not only the numerous unusually heavy rainfall events of the seasons of 1977-80 but also those of the period beginning in 1976 that led up to these unusual three seasons. All precipitation events listed in the table are single-station measurements.

The table lists for each event its date or dates, the name of the station at which the precipitation was measured, the amount of precipitation, the duration of the event, the computed return period, and other information, as outlined at the end of the table. Note that many of the return periods listed in the table are very large. For computed return periods much greater than the periods of record, the variance of the estimates becomes, of course, very large. Nonetheless, the large values of the return periods are retained in the table for purposes of comparison.

Two sets of sources for the statistical computations, including the return periods, were used. For stations in California the California Department of Water Resources Bulletin No. 195 (Goodridge, 1976a,b) was used: Volume I for durations up to and including 24 hours, Volume II for durations greater than 24 hours. Some interpolations between the statistics for the published durations and return periods were required. For events that occurred at stations for which return period computations are published in Bulletin No. 195, direct computations of the return periods were performed and tabulated, as were computations of the ratio of the event precipitation amount to the 100-year precipitation amount for the corresponding duration (column 6) and the number of standard deviations of the event precipitation above the mean for the corresponding duration (column 7). For these stations the number of years of precipitation record used in the computation of statistics in Bulletin No. 195 is also listed (column 1). For events that occurred at stations or locations for which return period statistics are not published in Bulletin No. 195, estimates of the return period of the event, plus estimates of the statistics tabulated in columns 6 and 7, were made based on statistics published in Bulletin No. 195 for nearby stations.

For stations in Arizona the National Weather Service publication <u>NOAA Atlas 2</u> (Miller et al., 1973) was used for precipitation statistics at durations up to and including 24 hours. The 2-, 3-, 4-, 7-, and 10-to-1-day ratios from the U.S. Weather Bureau (predecessor to the National Weather Service) publications Technical Paper No. 40 (Hershfield, 1961) and Technical Paper No. 49 (Miller, 1964) were used to extend the statistics computed from

TABLE 1 Some Extreme Precipitation Events in Southern California and Arizona, 1976-80 (see end of table for definitions of symbols; stations are in California except where noted)

Date(s)	Station	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1976									
February 8	Haines Canyon-Upper	36	LA	2.1	5 m	*	3.50	15.91	I
September 10	Mt. Laguna	4	SD	4.8	3 h	1,600	1.42	5.69	I
	Borrego County Road Station	12	SD	2.8	3 h	2,100	1.58	6.59	I
September 23	Borrego County Road Station	12	SD	4.0	3 h	*	2.26	10.09	I
	Hayfield Pumping Plant	33	R	3.87	12 h	4,800	1.74	7.45	I
October 1	Santa Maria Public Works	--	SB	1.2	17 m	*	1.76E	6.14E	I
	Goleta	--	SB	4.0	90 m	*	1.81E	7.36E	I
October 22	Jamul	--	SD	3.84	4 h	1,000E	1.40E	5.83E	I
1977									
August 15	Yuma Experiment Station,	--	WS	7.01	24 h	*	2.24E	9.98E	I
	Arizona					3,200	1.80	--	2
September 10	near Thousand Palms	--	R	4.5	2 h	*	2-3E	--	I
October 6-9	near Nogales, Arizona	--	WS	12.0	4 d	*	2.11	--	2
1977-78									
December 25-March 6	Opid's Camp	58	LA	71.64	71 d	155	1.04	3.25	II
December 25-March 31	Santa Barbara Flood Control District	--	SB	38.97	97 d	165E	1.08E	3.71E	II
1978									
January 3	Oceanside	24	SD	1.0	15 m	1,250	1.39	5.53	I
January 10	Santa Barbara Flood Control District	10	SB	1.22	15 m	3,300	1.51	5.87	I
January 16	Fallbrook	32	SD	1.2	15 m	1,000	1.36	5.38	I
				3.5	5 h	130	1.05	3.26	I

1978									
February 10	Haines Canyon-Upper	36	LA	1.4	30 m	50	.90	2.63	I
	Beverly Hills	--	LA	1.38	30 m	200E	1.10E	3.65E	I
February 28-March 2	Rock Springs, Arizona	--	GS	5.8	24 h	500	1.20	--	2
				7.2	48 h	800	1.29	--	2
				8.5	72 h	2,000	1.41	--	2
March 4	San Juan Guard Station	15	HP	1.9	1 h	2,000	1.46	5.89	I
				2.4	2 h	450	1.11	4.67	I
1979									
January 5	Newberry Park	16	V	3.04	4 h	72	.95	2.87	I
				3.69	6 h	70	.94	2.86	I
July 20	Cathedral City	7	R	1.37	30 m	*	2.54	11.70	I
				2.24	1 h	*	2.77	12.74	I
				2.92	2 h	*	2.52	11.47	I
				3.19	3 h	*	2.28	10.19	I
				3.68	6 h	*	1.89	8.21	I
November 7	Camarillo-Adohr	18	V	1.43	30 m	*	1.64	6.50	I
1980									
January 8-March 7	Cuyamaca	87	CD	51.91	60 d	110	1.01	3.07	II
	Palomar Mountain Observatory	36	CD	44.31	60 d	280	1.12	3.66	II
February 14-22	Crown King, Arizona	--	CD	16.49	9 d	1,200	1.46	--	2
	Indio State Division of Forestry	24	R	4.35	8 d	820	1.37	5.37	II
	Indio Date Garden	73	CD	3.89	8 d	55	.88	2.89	II
	Elsinore	26	R	12.34	8 d	370	1.18	4.07	II
February 15	Los Angeles-Hancock Park	42	LA	1.0	30 m	33	.84	2.32	I
February 16	Fillmore Fish Hatchery	18	V	4.88	6 h	145	1.05	3.43	I
	Sepulveda Dam	29	CE	4.57	5 h	165	1.08	3.48	I
				5.70	8 h	160	1.06	3.42	I
				5.98	9 h	160	1.06	3.43	I
February 20	Flinn Springs	11	SD	2.05	3 h	45	.87	2.67	I
March 4	El Capitan Dam	18	SD	1.0	15 m	140	1.04	3.68	I
September 9	near Franklin, Arizona	--	WS	4.0+	3 h	2,000+	1.48+	--	2

Note: Explanations of symbols.

(1) Number of years of record.
(2) Source of 1976-80 precipitation data:
- CD = Climatological Data (U.S. National Oceanic and Atmospheric Administration, 1976-80a).
- CE = U.S. Army Corps of Engineers.
- GS = U.S. Geological Survey.
- HP = Hourly Precipitation Data (U.S. National Oceanic and Atmospheric Administration, 1976-80b).
- LA = Los Angeles County Flood Control District.
- R = Riverside County Flood Control and Water Conservation District.
- SB = Santa Barbara County Flood Control and Water Conservation District.
- SD = San Diego County Department of Sanitation and Flood Control.
- V = Ventura County Flood Control District.
- WS = Unpublished data provided by National Weather Service.

(3) Precipitation amount in inches.
(4) Duration of precipitation event (m = minutes, h = hours, d = days).
(5) Estimated return period in years.
(6) Ratio of event precipitation amount to the 100-year precipitation for the corresponding duration.
(7) Number of standard deviations above the mean.
(8) Source of precipitation statistics:
- I = Bulletin No. 195, Volume I (Goodridge, 1976a).
- II = Bulletin No. 195, Volume II (Goodridge, 1976b).
- 2 = NOAA Atlas 2 (Miller et al., 1973) for durations to 24 hours. 2-, 3-, 4-, 7-, and 10-to-1-day ratios from Technical Paper No. 40 (Hershfield, 1961) and Technical Paper No. 49 (Miller, 1964).

* Estimated return period $\geq$ 10,000 years, according to published statistics.

-- Not available.

\+ Gage overflowed. Value listed may have been exceeded.

E Value listed is an estimate.

the 24-hour values of NOAA Atlas 2. Since the return period computations in these U.S. Weather Bureau and National Weather Service publications are performed entirely on a regional basis instead of on an individual station basis, no tabulations of the number of years of record for individual stations (column 1) or for standard deviations (column 7) are available.

At the Yuma Experiment Station in Arizona, located very near the Colorado River border between California and Arizona, statistics are available both in Bulletin No. 195 for the nearby Yuma Airport and in NOAA Atlas 2. Thus for the August 15, 1977, precipitation event at the Yuma Experiment Station, the return period and other statistics have been computed, using both Bulletin No. 195 and NOAA Atlas 2, with understandably somewhat differing results.

A discussion of some of the unusual rainfall events listed in the table will follow later in this paper.

OCEANIC AND METEOROLOGICAL CONDITIONS

Some of the oceanic and meteorological conditions that were responsible for the unusually heavy precipitation during the years 1977-80 have already been discussed in the papers by Jerome Namias and by Carlos Garza and Craig Peterson. As noted by these authors, the primary atmospheric pattern characteristic of these three seasons, especially those of 1977-78 and 1979-80, was a low-latitude jet stream and storm track that impinged upon southern California and Arizona from the west and southwest. This pattern appears to have resulted, at least in part, from anomalous sea surface temperatures over the North Pacific. A trough of low pressure west or southwest of southern California also tended to recur persistently during these years, including the 1978-79 season. The circulation around this low periodically brought large quantities of warm and very moist tropical air into the southwestern United States from out of the equatorial and subequatorial Pacific Ocean south and southwest of the region. It was in these surges of unusually moist air flow that a number of the extremely heavy short- and intermediate-duration precipitation events of 1977-80 occurred.

COMMENCEMENT OF PATTERN

Portions of the atmospheric patterns that led to the unusual concentration of heavy precipitation events during 1977-80 actually began to develop in early 1976, while California and other western states were in the midst of a rapidly worsening drought--a drought that was to be the most severe of recent decades. In early 1976 the low-latitude storm track from across the Pacific had not yet developed. On the contrary, an extremely large ridge of high pressure persisted on the average over the far western United States, sending the Pacific storms far to the north of the region. The drought that persisted from 1975 through much of 1977 was extremely severe in northern and central California and only somewhat less severe in southern California and Arizona. What precipitation these southern areas did receive during the middle and latter part of the drought occurred as the result of an occasional low-latitude upper-level trough of low pressure that developed over or just west of southern California. This trough appears to have been the forerunner of the trough that persistently recurred off the southern California coast during the 1977-80 seasons.

Even though the average seasonal precipitation in southern California and Arizona during the drought years of 1975-77 was limited, the presence of the recurring upper-level trough, and the influx of moist tropical air into the southwestern United States in the circulation around the trough, resulted in several locally very heavy precipitation events in southern California and Arizona, beginning in early 1976. For this reason the listing of extreme precipitation events for the 1977-80 rainfall seasons has been expanded backward to include the major events beginning in early 1976.

WINTER AND SUMMER EVENTS

It can be seen in Table 1 that most of the unusually heavy precipitation events listed, including all of the events having durations of more than four days, occurred during the winter months, December through March. There are, however, a few very prominent summer and fall events that occurred, primarily during 1976 and 1977. The major reason for this appears to be the fact that the upper-level trough off the southern California coast and the associated influx of very moist tropical air into the southwestern United States were not confined to the winter months but instead persisted on and off during the warmer seasons of the year. Several of the major summer and early fall events listed in the table occurred as the direct result of this recurring trough, while others occurred as the consequence of tropical storms from off the west coast of Mexico being caught up in the circulation around this upper-level trough and rapidly steered unusually far northward into southern California or Arizona. The events of September 10, 1976, August 15, 1977, and October 6-9, 1977, were all directly associated with such tropical storms.

DISCUSSION

It can be seen in Table 1 that all of the precipitation entries listed are considered rare events. Only those events having return periods of approximately 40 to 50 years or greater were selected for this table. Many of the events exceed 1,000 years in computed return period, according to the available statistics, and a few appear greatly to exceed 10,000 years. (An event with a return period of 10,000 years should have approximately a one percent chance of occurring some time during the next 10 years.)

Of course, considerable argument can be advanced about computed return period statistics such as those listed in the table. For one thing, not enough is known about the nature and magnitudes of precipitation in very major storm events or about the statistical distributions of such events compared with those of the more common storm events. It may be that very large summer thunderstorms, tropical storms, and even some of the giant winter rainstorms that occasionally affect southern California and Arizona should rightfully not be considered part of the same population as the lesser storms upon which the records and the computed return period statistics must often be based. No one really knows the answers to these questions.

For another thing, even if the true statistical distribution(s) of the precipitation population(s) in southern California and Arizona was (were) known, the precipitation records at stations in the southwestern United States are not nearly long enough for one to be able to determine 1,000-year, let alone 10,000-year, return periods meaningfully. The case of Cathedral City on July 20, 1979, illustrates this point very graphically. In the seven years of record (1969-75 inclusive) used to determine the return period statistics, no large-magnitude storm had occurred. If the 1976-80 seasons had been included in the record used in Bulletin No. 195, the wet years of 1977-80 in general and the unusually heavy September 1976 and July 1979 storms in particular would have greatly lowered the return periods for the July 20, 1979, event as listed in Table 1.

A similar situation is demonstrated in the November 7, 1979, Camarillo-Adohr event, where return period computations published in Bulletin No. 195 based on the 18-year period of record, 1957-74 inclusive, would indicate a return period for the event of more than 10,000 years, while computations based on a period of record through the very latest available data, including the November 1979 event itself, indicate a return period for that event of only 1,300 years, according to the Ventura County Flood Control District.

Another interesting case can be seen in Indio, California, during the period February 14-22, 1980. At the Indio Date Garden (73 years of record) all precipitation statistics, as published in Bulletin No. 195, from the mean of the annual n-day maximums (where n ranges from 1 to 365) all the way to the computed 10,000-year and probable maximum precipitation values, are significantly higher than those published for the Indio State Division of Forestry (SDF) station. Thus for a precipitation event of a given magnitude at both Indio stations, the computed return period should be smaller at the Date Garden than at the SDF station. In the February 1980 storm period it turns out that the SDF station (normally the drier station) was significantly wetter in February 1980 than the Date Garden. Thus the computed return period for the eight-day 1980 event is very much greater at Indio SDF than at the Date Garden.

Although some of the return periods in the table do appear to be quite extreme, it can safely be stated that the gages that measured the rainfall amounts listed in the table did not always sample the heaviest rainfall of the storm. Such is known to be the case on the morning of February 15, 1980, where the 1.0 in. measured in 30 minutes at Los Angeles-Hancock Park exhibits a return period of only 33 years. It turns out, however, from the qualitative observations of the author and those of some of his colleagues that several areas farther to the west, located between recording rain gages, experienced significantly heavier precipitation for durations up to around one hour. It is felt that such intensities, if measured, would have easily exceeded, and perhaps considerably exceeded, those for the 100-year rainfall.

Another case where the center of heaviest rain may not have been officially sampled occurred just one day later on the morning and afternoon of February 16, 1980. In the table it can be seen that the Fillmore Fish

Hatchery and Sepulveda Dam recorded precipitation over durations of five to nine hours that exhibited computed return periods in the vicinity of 150 years. However, there may have been even heavier rainfall in Topanga Canyon (in the mountains to the south of the two listed stations), where extreme flooding and mudslide damage were reported. One unofficial and unsubstantiated amount of 10.7 in. in 10 hours was reported to the National Weather Service, but no confirmation of this claim could be made. If this amount and duration were true, it would represent a return period of approximately 600 years or perhaps greater, depending just where in Topanga Canyon (with its highly orographic variations) the gage had been located.

It should be pointed out that this table lists one truly extreme precipitation event. The very first entry--that of February 8, 1976, at Haines Canyon-Upper--is an all-time State of California record for five-minute rainfall. The amount and duration are official and confirmed, according to the Los Angeles County Flood Control District, although some meteorologists feel that there still might be some question as to the accuracy of this recording gage's measurement. If the value is correct, it is 3.5 times the 100-year value for the same duration, according to Bulletin No. 195, and the number of standard deviations above the mean for this event is an incredible 15.91--thus exceeding the computed probable maximum precipitation, which is defined in Bulletin No. 195 as the mean plus 15 standard deviations.

CONCLUSIONS

In examining the table in this paper one might reach the conclusion that the years 1976-80 were truly extreme in southern California and Arizona with regard to precipitation. With the large number of events whose estimated return periods exceed 1,000 and even 10,000 years, it could perhaps be surmised that such a five-year period was exceptionally rare throughout the southwestern United States. This, of course, is not true. In almost any year, with the large number of statistically independent rain gages that there are in southern California and Arizona (gages that are especially independent from each other with respect to high-intensity local downpours), there are likely to be at least one or two precipitation events somewhere in the region that have return periods of more than 100 years, and possibly one or more in some years with return periods of 1,000 to 10,000 years or so.

Nevertheless, the number of major events recorded during the years 1976-80 is significantly larger than that of a similar period of time taken at random. Thus it would appear that the past few years have been very much more active than normal as far as rare rainfall events are concerned. This obviously has had a major bearing upon the large number of floods, especially flash floods, and mudslides that have been observed in southern California and Arizona from late 1976 through much of 1980.

REFERENCES

Goodridge, J. D. (1976a) Rainfall Analysis for Drainage Design, Volume I: Short-Duration Precipitation Frequency Data, Bulletin No. 195, California Department of Water Resources, Sacramento.

Goodridge, J. D. (1976b) Rainfall Analysis for Drainage Design, Volume II: Long-Duration Precipitation Frequency Data, Bulletin No. 195, California Department of Water Resources, Sacramento.

Hershfield, D. M. (1961) Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years, Technical Paper No. 40, U.S. Weather Bureau, Department of Commerce, Washington, D.C.

Miller, J. F. (1964) Two- to Ten-Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States, Technical Paper No. 49, U.S. Weather Bureau, Department of Commerce, Washington, D.C.

Miller, J. F., Frederick, R. H., and Tracey, R. J. (1973) Precipitation-Frequency Atlas of the Western United States, Volume VIII, Arizona, NOAA Atlas 2, U.S. Department of Commerce, Silver Spring, Maryland.

U.S. National Oceanic and Atmospheric Administration (1976-80a) Climatological Data, monthly issues for Arizona and California, U.S. Department of Commerce, Silver Spring, Maryland.

U.S. National Oceanic and Atmospheric Administration (1976-80b) Hourly Precipitation Data, monthly issues for Arizona and California, U.S. Department of Commerce, Silver Spring, Maryland.

HYDROLOGY OF THE FLOODS OF MARCH 1978 THROUGH FEBRUARY 1980 IN THE PHOENIX AREA, ARIZONA

by B. N. Aldridge

Severe precipitation in the mountains north and east of Phoenix caused five floods in the Phoenix area from March 1978 through February 1980. The floods occurred in March 1978, December 1978, January 1979, March 1979, and February 1980 when the flow in the Salt, Verde, and Agua Fria rivers exceeded the storage capacity of the reservoirs on the rivers. In most years the storage capacity of the reservoirs is sufficient to store the inflow. Water has been released nine times in 70 years on the Salt River, six times in 40 years on the Verde River, and five times in 53 years on the Agua Fria River; the largest peak release rates were during the floods of 1978-80.

The reservoirs were filled in March 1978 by the large runoff volumes from more than five days of heavy precipitation. The seven-day floodflow was the maximum of record in the Verde River and Tonto Creek and the second highest of record in the Salt River. The reservoirs were filled again in December 1978 and in February 1980 because of large amounts of inflow and large carryover storage.

Hydrographs of the floods along the Salt and Verde rivers do not show the typical progression of a floodwave moving downstream. Flood crests occurred more or less concurrently at all sites along the rivers. Tributary inflow caused the crests at some downstream stations to precede the crests at the upstream stations; this complex runoff pattern restricted advance warning of the magnitude of the floods approaching the reservoirs.

The storage provided by the reservoirs greatly reduced the magnitude and duration of the floods in Phoenix. Without the reservoirs the peak discharges of the Salt River would have been as much as twice the actual discharges, and high flows would have lasted for several days.

Severe precipitation in the mountains north and east of Phoenix caused five floods in the Phoenix area from March 1978 through February 1980. Floods occurred in March 1978, December 1978, January 1979, March 1979, and February 1980 when the flow in the Salt, Verde, and Agua Fria rivers exceeded the storage capacity of the reservoirs on the rivers. The Salt River flows

---

B. N. Aldridge is Hydrologist with the U.S. Geological Survey in Tucson, Arizona.

westward through the central part of the area, and the Agua Fria River flows southward along the west side; the rivers join the Gila River west of Phoenix. River basins that contributed to the floods and the main streamflow-gaging stations are shown in Figures 1 and 2, respectively.

The peak discharge of 122,000 cu ft/s in March 1978 was the highest since 1920 in the Salt River at Jointhead Dam, Phoenix, and the flood caused $37 million in damage in Maricopa County (U.S. Army Corps of Engineers, 1979a). The area was just recovering from the flood of March 1978 when another major flood occurred in December 1978. The peak discharge of 141,000 cu ft/s in December 1978 was the highest since 1905 in the Salt River at Jointhead Dam, and the flood caused $51.8 million in damage in the Phoenix area (U.S. Army Corps of Engineers, 1979b). Smaller floods in January and March 1979 caused little additional damage except for closing an interstate highway bridge that survived the flood of December 1978 and delaying the reopening of other river crossings; the peak discharges for these floods were about 85,000 and 60,000 cu ft/s, respectively. The area again was recovering when an even larger flood occurred in February 1980. The peak discharge was 170,000 cu ft/s at Jointhead Dam, and the flood damage is still being assessed. The last time that floods occurred so frequently in the Phoenix area was when nine flows of more than 50,000 cu ft/s occurred between February 1905 and March 1906. The floods of March 1978, December 1978, and February 1980 also caused considerable damage outside the Phoenix area. Damages in outlying areas from the floods of March and December 1978 were $29.8 and $39.8 million, respectively; damage from the flood of February 1980 is still being assessed.

More than 60,000 cu ft/s was released from Lake Pleasant to the Agua Fria River during the flood of December 1978 and again during the flood of February 1980. Downstream from Lake Pleasant the peak discharges were the highest since 1919, and large areas were inundated (Thomsen, 1980). The floods of March 1978, January 1979, and March 1979 on the Agua Fria River downstream from Waddell Dam caused some damage and inconvenience but were not hydrologically significant.

The primary purpose of the reservoirs upstream from Phoenix is to store water for irrigation use, but the reservoirs also provide considerable flood protection. Reservoir storage capacity greatly reduced the magnitude and duration of the floods of 1978-80. Without the reservoirs the peak discharge of the Salt River during the flood of March 1978 would have been about 260,000 cu ft/s (Figure 3), and the peak discharge during the flood of December 1978 would have been about 240,000 cu ft/s. The reduction in peak discharge was less dramatic during the flood of February 1980 but was sufficient to reduce greatly the flooding in Phoenix. The reservoirs did not significantly reduce the floods of January and March 1979 because the reservoirs were full owing to the flood of December 1978.

Flood control provided by the reservoirs is the result of large capacities and operating procedures. The capacities of the reservoirs on the Salt and Agua Fria rivers are considerably greater than the median annual streamflow (Table 1). On the average, annual streamflow exceeds reservoir capacities about 1 year in 12 in the Salt River and 1 year in 30 in the Agua Fria River.

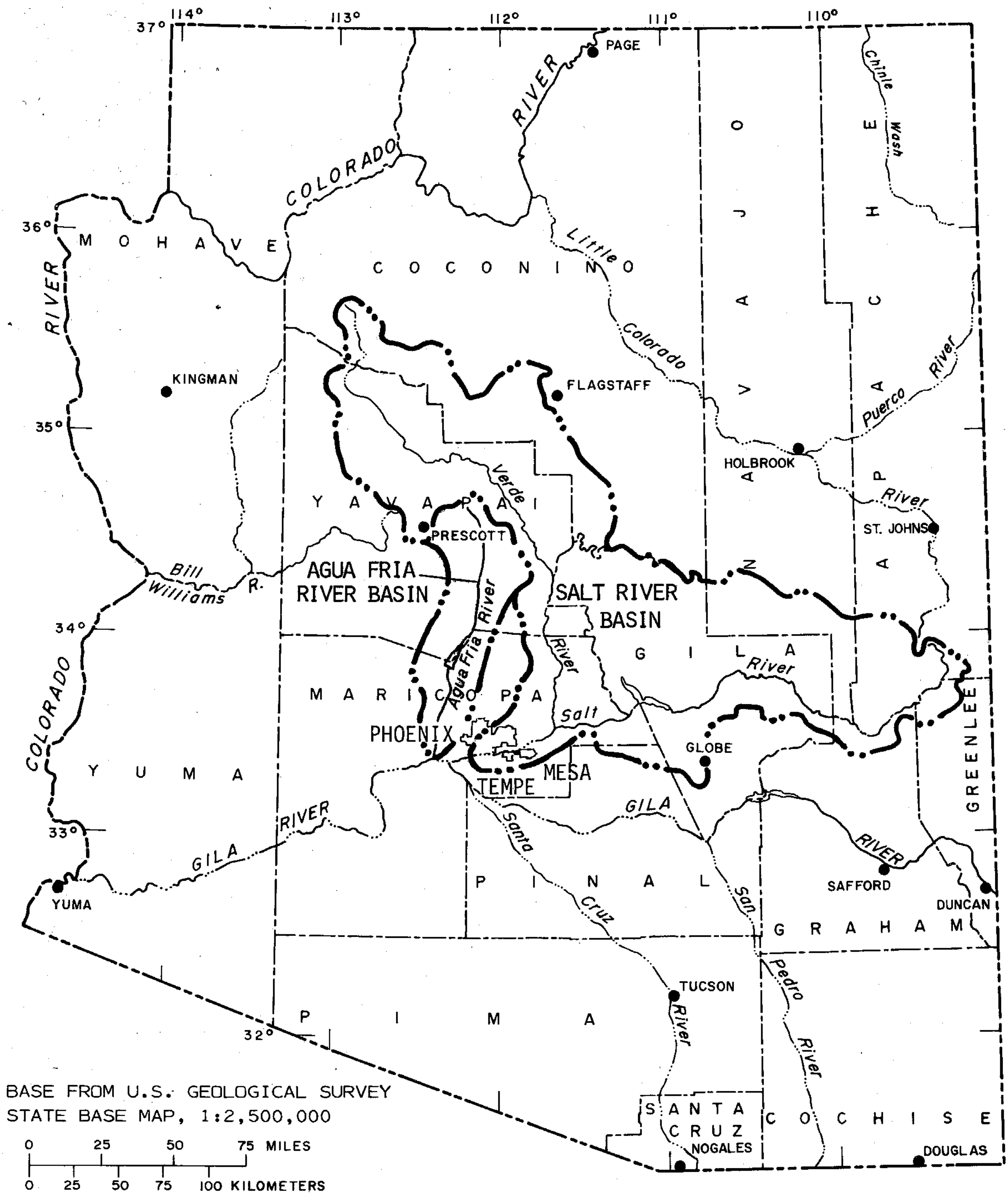
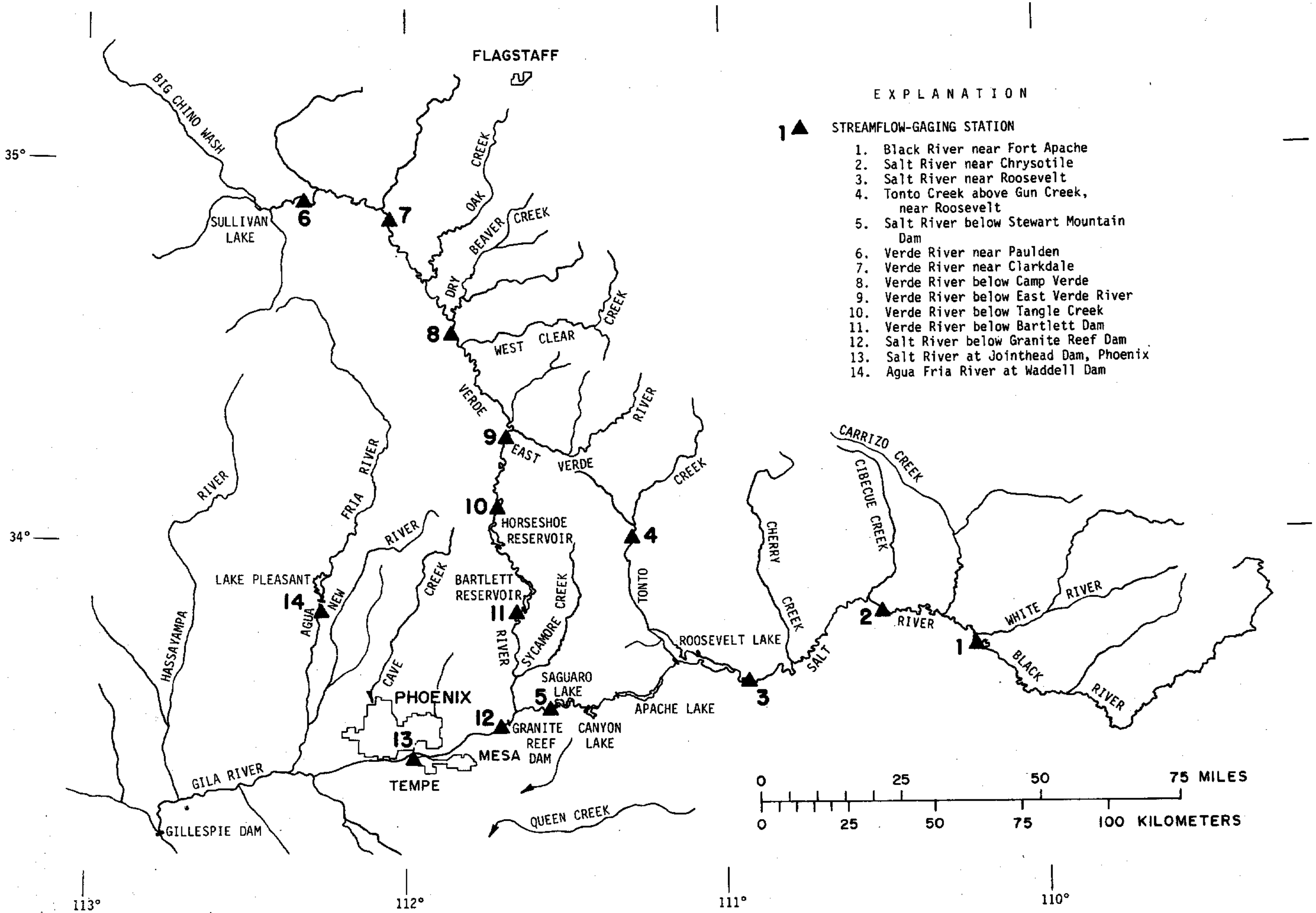


FIGURE 1 River basins that contributed to the floods of March 1978 through February 1980 in the Phoenix area.



BASE MAP FROM U.S. GEOLOGICAL SURVEY
STATE BASE MAP, 1:1,000,000

FIGURE 2 Main streamflow-gaging stations in the Salt River and Agua Fria River basins.

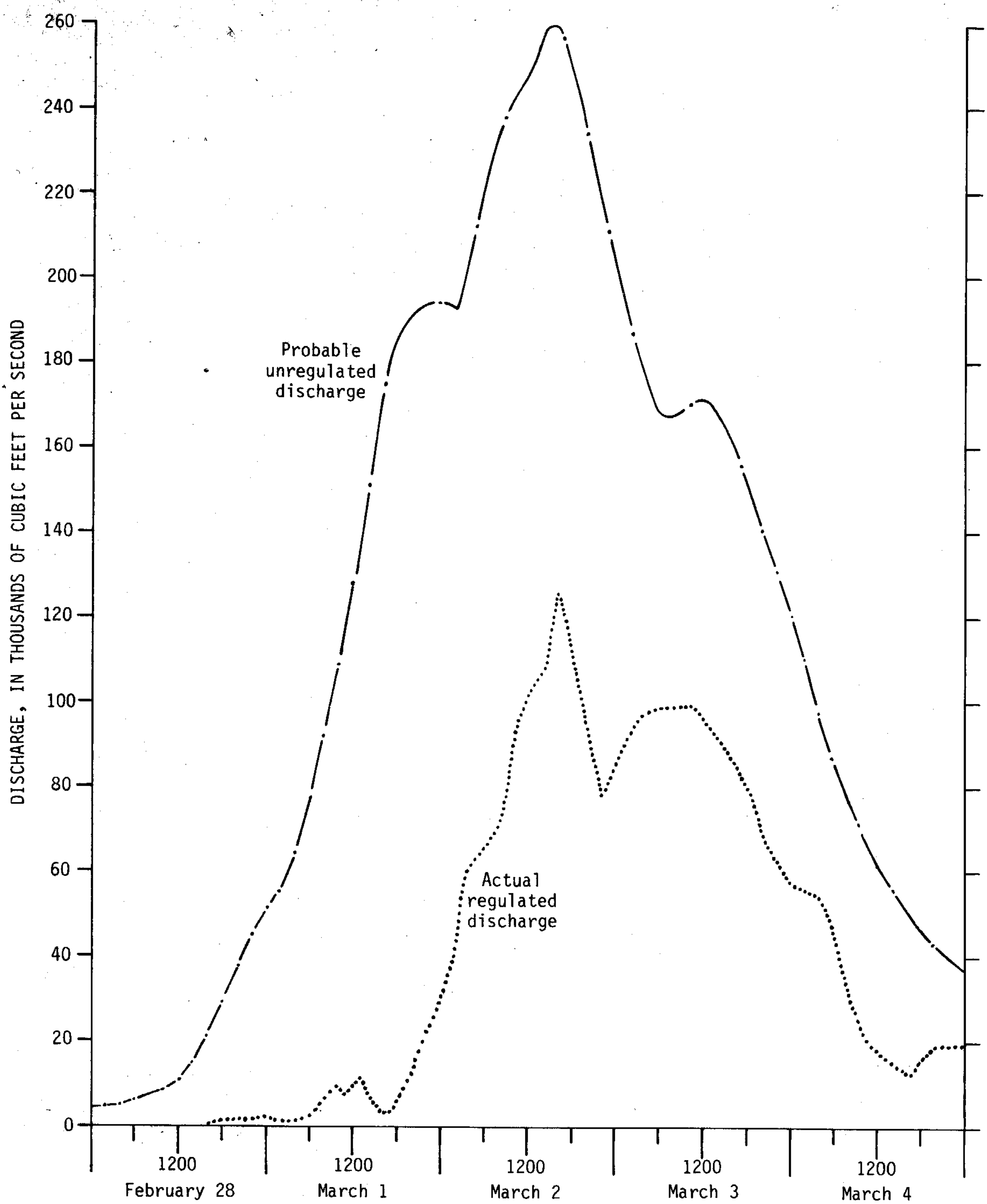


FIGURE 3 Comparison of actual regulated and probable unregulated discharge for the flood of March 1978, Salt River below Granite Reef Dam.

TABLE 1 Minimum, Median, and Maximum Annual Streamflow and Reservoir Capacities, Salt, Verde, and Agua Fria Rivers (acre-ft)

	Salt River	Verde River	Agua Fria River
Streamflow			
Minimum	189,000[a]	137,000[b]	3.000[c]
25th percentile	317,000[a]	193,000[b]	11,000[c]
Median	530,000[a]	240,000[b]	24,000[c]
75th percentile	822,000[a]	410,000[b]	60,000[c]
Maximum	2,731,000[a]	1,238,000[b]	221,000[c]
Reservoir capacities	1,755,000	317,000	157,000

[a]Summation of streamflow, Salt River near Roosevelt and Tonto Creek (near Roosevelt, 1914-41; above Gun Creek near Roosevelt, 1941-78). Represents at least 95 percent of the inflow to Roosevelt Lake.

[b]Based on records from several sites between Tangle Creek and the mouth of the Verde River, 1905-78. Represents inflow to Horseshoe Reservoir.

[c]Computed inflow to Lake Pleasant, 1914-19, 1933-78.

In the Verde River the annual streamflow exceeds reservoir capacities about 1 year in 3; however, during much of the year the streamflow is released almost immediately from the reservoirs, thereby increasing the potential for flood protection. The degree of flood control provided by the reservoirs during a flood depends on the amount of carryover storage from preceding years. The carryover storage shown in Figure 4 is the storage that was carried over after the irrigation season in the calendar year shown. Water is released over the spillways only when the reservoirs are nearly full and the inflow is greater than the amount needed for irrigation. Water has been released nine times in 70 years on the Salt River, six times in 40 years on the Verde River, and five times in 53 years on the Agua Fria River. The floods of December 1978 and February 1980 are examples of the effects of a sharp peak superimposed on a high carryover storage, and the flood of March 1978 is an example of the effect of a large volume of runoff on reservoir storage. Several hydrologic characteristics of the flood of March 1978 are important to the understanding of floods in the Salt River basin, and the flood is analyzed more extensively in this paper than the other floods.

The runoff volumes during the flood of March 1978 were the largest since the beginning of record in 1905 for the Verde River and in 1914 for Tonto Creek (Table 2). The volume of runoff in the Salt River was the largest since 1916. Inflow from ungaged tributaries downstream from the main gaging stations on the Salt River, Tonto Creek, and Verde River was the largest computed since the reservoirs were constructed.

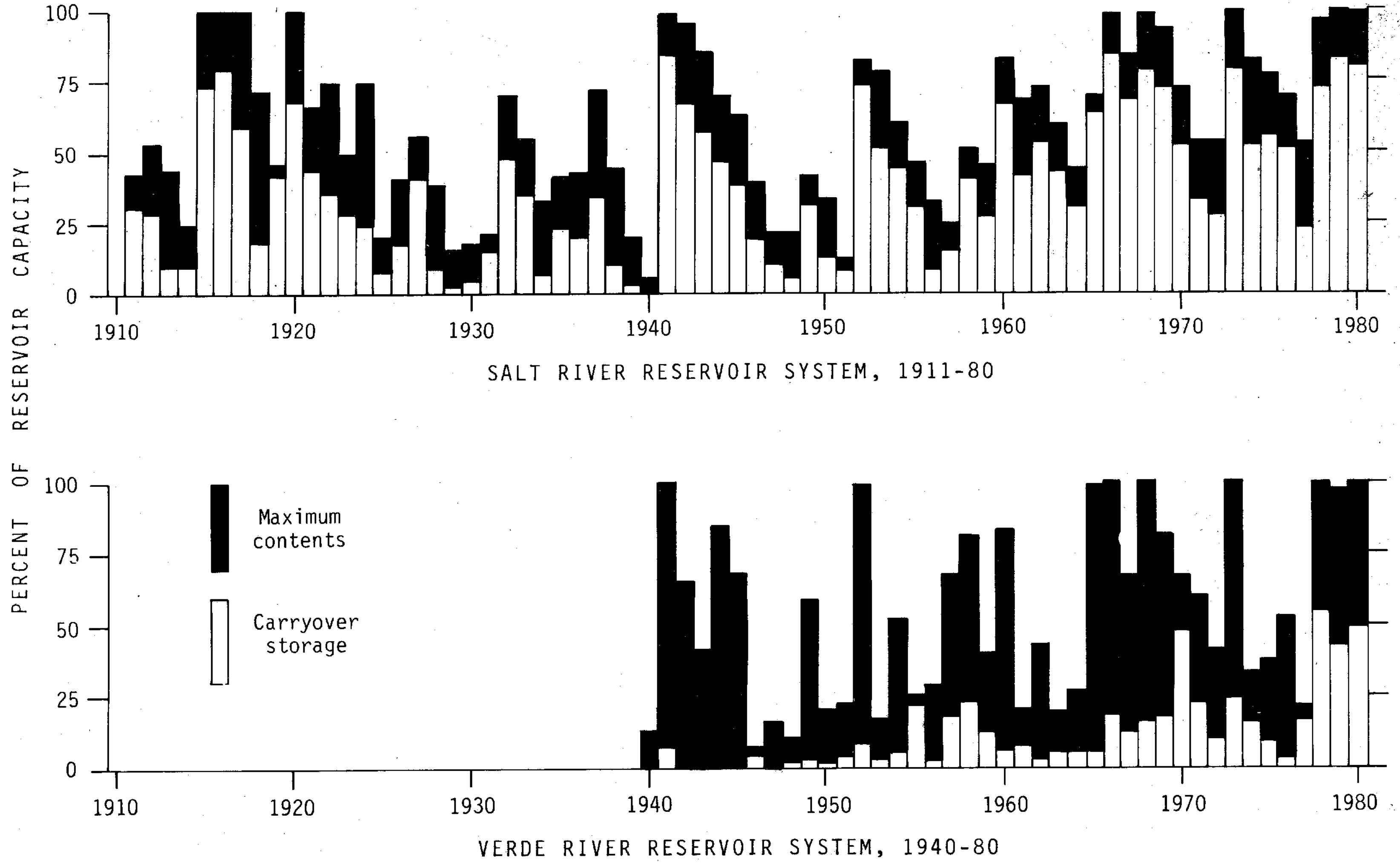


FIGURE 4 Spring contents and carryover storage at the end of the irrigation season, Salt and Verde reservoir systems.

TABLE 2 Highest Three- and Seven-Day Mean Discharge, at Selected Gaging Stations, Salt River, Tonto Creek, and Verde River (cu ft/s)

	Salt River near Roosevelt, 1915-80	Tonto Creek above Gun Creek, 1914-80[a]	Verde River below Tangle Creek, 1905-80[b]
Prior to March 1978			
Three-day mean			
Highest discharge	71,800	12,400	42,100
Year	(1916)	(1916)	(1916)
2nd highest discharge	46,600	10,700	40,800
Year	(1941)	(1951)	(1927)
Seven-day mean			
Highest discharge	42,400	7,630	23,100
Year	(1916)	(1916)	(1927)
2nd highest discharge	24,600	5,930	22,500
Year	(1941)	(1924)	(1920)
March 1978-February 1980			
Three-day mean			
March 1978	59,200	27,200	61,500
December 1978	44,800	13,100	39,000
January 1979	20,700	11,200	12,900
March 1979	12,900	6,020	9,020
February 1980	35,000	16,300	34,100
Seven-day mean			
March 1978	32,100	15,400	34,900
December 1978	22,500	6,090	18,700
January 1979	11,100	5,550	7,170
March 1979	9,310	3,850	6,740
February 1980	25,600	11,800	30,900

[a]Based on records from Tonto Creek near Roosevelt, 1914-41; above Gun Creek near Roosevelt, 1941-78.

[b]Based on records from several sites between Tangle Creek and the mouth of the Verde River.

These large volumes of runoff occurred in a series of crests and troughs in a three-day period--March 1-3--followed by a small rise on March 5. The shapes of the hydrographs and the times of the maximum crests at the stations were not consistent. The orographic effect of the mountains caused localized rainfall patterns, differences in altitude and exposure caused variations in snowmelt, and the direction of storm movement relative to the direction of flow caused variations in the timing of flood crests.

Timing of the crests on the tributaries significantly affects the shape of the floodwave and the magnitudes of the crests on the Salt and Verde rivers. Hydrographs of floods along these rivers seldom show the typical progression of a floodwave moving downstream. Generally the flood crests occur more or less concurrently at all sites along the rivers because the tributary inflow adds progressively more discharge at each downstream station. Frequently, as during the flood of March 1978, tributary inflow is sufficient to cause a flood to crest at a downstream station before the flood has crested at an upstream station.

The crest of the flood of March 1978 at the Salt River near Roosevelt gaging station was the result of large tributary inflow in addition to the increasing discharge of the Salt River (Figure 5). The crest occurred several hours before the crest at Salt River near Chrysotile, the next upstream station. The crest at the station near Chrysotile caused a flattening of the recession at the station near Roosevelt. No crest of the Salt River near Roosevelt can be identified with a corresponding crest of the Black River near Fort Apache. Tonto Creek and the other tributaries to Roosevelt Lake crested within a few hours of the Salt River. The nearly simultaneous crests produced a single high crest of inflow to the lake.

Figure 6 shows the effects of tributary inflow to the Verde River during the flood of March 1978. The rapid rise of the Verde River at the gaging station below Tangle Creek was concurrent with the rise at the station below East Verde River and preceded by several hours the rise at the station below Camp Verde. At 1200 hours on March 1, when the most rapid rise (crest A) began below Camp Verde, the Verde River below Tangle Creek was already cresting at 80,000 cu ft/s because of the large tributary inflow between the stations. Crest A can be traced from below Camp Verde to below Tangle Creek. Crest B, which occurred a few hours later, originated downstream from the East Verde River. Crest C originated between Camp Verde and the East Verde River and probably resulted from the same surge of rainfall as crest B; however, crest C lagged behind crest B because of the greater distance of travel. Crests D and E can be identified at the three upstream stations but not at the two downstream stations. Rises that may have occurred owing to the crests were obscured by tributary inflow from later rainfall. Crest D probably reached the two downstream stations concurrently with crest C. The flattening on the rising limb of the crest on March 3 may have been the result of crest E. Crests D and E resulted from the same period of rainfall; however, crest E was delayed because of storage in Sullivan Lake, the circuitous route followed by tributaries to Big Chino Wash, and a slow travel time along the wash, which has a much flatter slope than most streams in the Verde River basin.

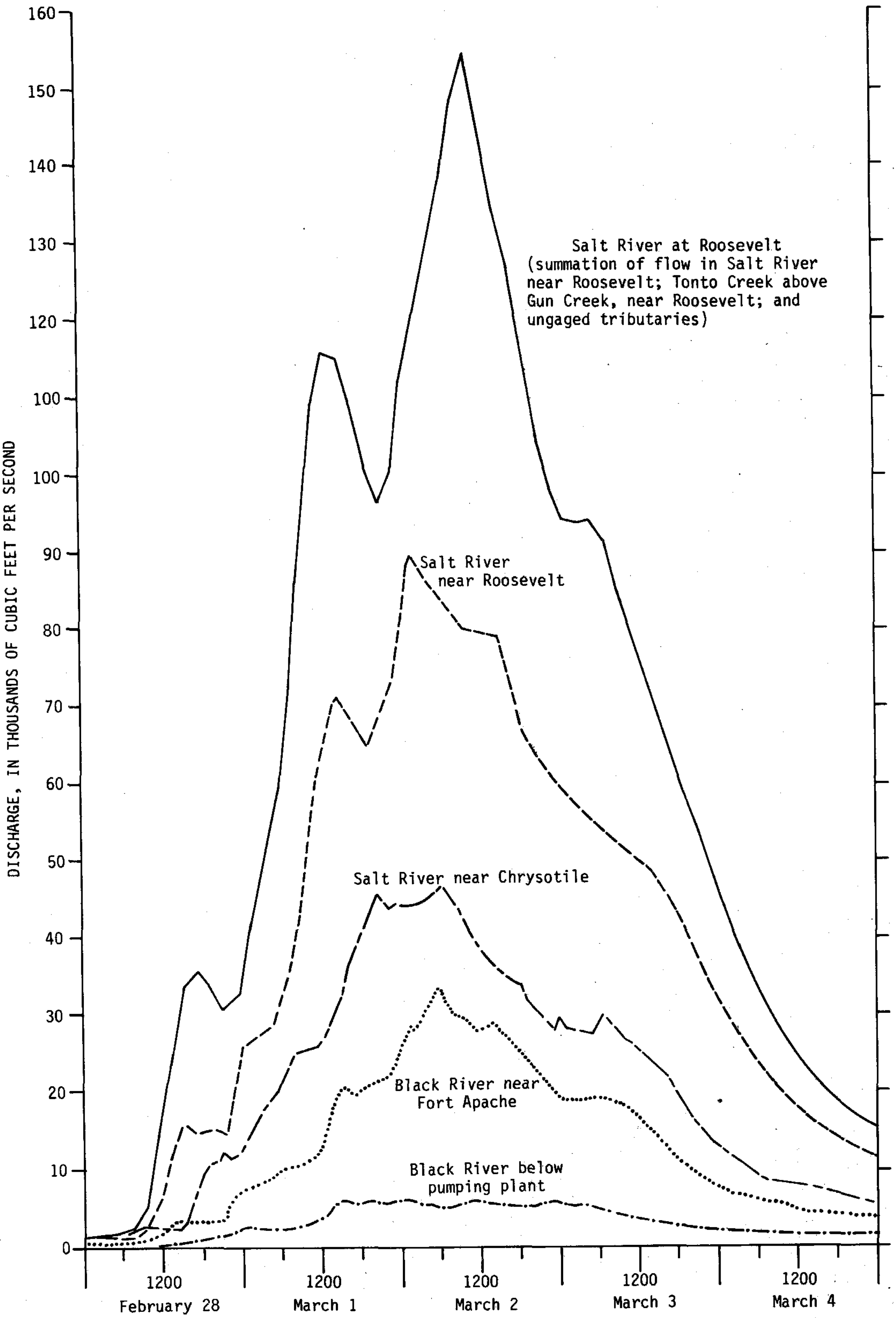


FIGURE 5 Discharge of the Black and Salt rivers upstream from Roosevelt Dam during the flood of March 1978.

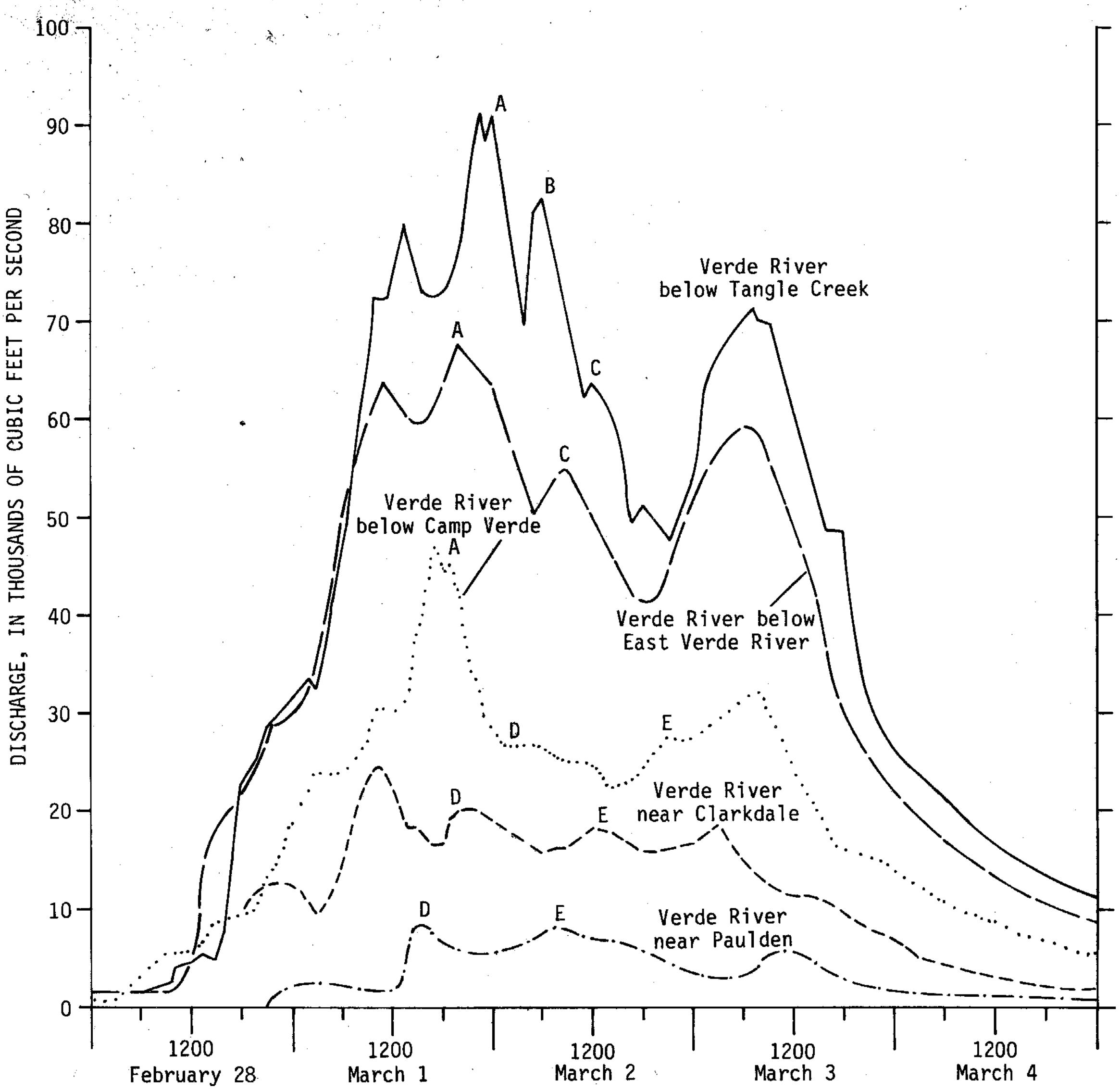


FIGURE 6 Discharge of the Verde River during the flood of March 1978.

The complex runoff pattern in the Verde River basin restricted advance warning of the magnitude of the flow approaching the reservoirs during the flood of March 1978. The only advance warning was that provided by the gaging station below Tangle Creek, which is only 4 miles upstream from Horseshoe Reservoir. Although the short flood warning complicated the scheduling of releases from the reservoirs, the maximum release rate was kept within a few percent of the maximum inflow rate.

During the first two days of the flood, the release rates were kept well below the inflow rates to the reservoirs. A critical situation developed about 1000 hours on March 2, when about 60,000 cu ft/s was being released from Bartlett Reservoir. The reserve storage capacity in the reservoir system was down to 13,400 acre-ft. Inflow to Horseshoe Reservoir was about 68,000 cu ft/s, or about 5,400 acre-ft/hr, and the record for the gaging station below East Verde shows that the river was beginning to rise for the fourth time in about 24 hours; each crest had the potential of being higher than the preceding ones. Concurrently, the ungaged tributaries were discharging large but unknown quantities of water directly into the reservoirs; subsequent analyses show that the tributary inflow probably was between 20,000 and 30,000 cu ft/s. Between 1000 and 1100 hours the outflow was increased from 60,000 cu ft/s to about 100,000 cu ft/s, which was a few percent greater than the total inflow. The rise at the gaging station below East Verde did not materialize as a significant peak at the station below Tangle Creek, and outflow was reduced to 65,000 cu ft/s by 1230 hours.

Although the release from Bartlett Reservoir caused most of the floodflow in Phoenix in March 1978, some water was released at Stewart Mountain Dam, the outlet of Saguaro Lake, on the Salt River owing to tributary inflow downstream from Roosevelt Dam; no water was released at Roosevelt Dam.

In contrast, large quantities of water were released at Roosevelt Dam during the floods of December 1978, January 1979, March 1979, and February 1980 because of the large amount of carryover storage from preceding floods. Water released from Roosevelt Lake passed directly through Apache, Canyon, and Saguaro lakes and was released at Stewart Mountain Dam. During the floods of December 1978 and February 1980, about equal amounts of water were released from Bartlett and Stewart Mountain dams. The floods of January and March 1979 were caused mainly by the releases from Stewart Mountain Dam.

Although the inflow volumes to Roosevelt Lake during the floods of December 1978 and February 1980 were among the five largest of record, the flows would not have necessitated spillway releases if the carryover storage had not been so large. Carryover storage is low enough 80 percent of the time to permit storage of inflow volumes equal to those of the December 1978 or February 1980 floods. The large inflow volumes to the reservoirs on the Verde River would have necessitated some release even if the carryover storage had been at the average level. The floods of December 1978 and February 1980 were single high crests, whereas the flood of March 1978 consisted of multiple peaks.

REFERENCES

Thomsen, B. W. (1980) Flood of February 1980 along the Agua Fria River, Maricopa County, Arizona, Open-File Report 80-767, U.S. Geological Survey, Tucson, Arizona, map.

U.S. Army Corps of Engineers (1979a) Flood Damage Report on the Storm and Floods in Maricopa County, Arizona, 28 February-6 March 1978, U.S. Army Engineers District, Los Angeles.

U.S. Army Corps of Engineers (1979b) Flood Damage Report, Phoenix Metropolitan Area, December 1978 Flood, U.S. Army Engineers District, Los Angeles.

U.S. Army Corps of Engineers (1980) Flood Damage Report, Southcentral Arizona and Southwestern New Mexico, December 1978 Flood, U.S. Army Engineers District, Los Angeles.

U.S. National Oceanic and Atmospheric Administration (1978) Arizona Flooding, February-March 1978, Arizona Technical Memorandum AZ8, Salt Lake City.

U.S. National Oceanic and Atmospheric Administration (1979) Report on the Arizona Floods of December 16-20, 1978, Salt Lake City.

FLOODS OF JANUARY AND FEBRUARY 1980 IN CALIFORNIA

by Kenneth L. Wahl, John R. Crippen, and James M. Knott

During January and February 1980 storms caused substantial rises in streamflow throughout much of California. In mid-January flooding occurred in the foothills of the Sierra Nevada and in the central coastal area. In late January and mid-February high floodflows in streams in coastal southern California caused much damage and several deaths. The Tijuana River in northern Baja California (Mexico) and southern San Diego County flooded many square miles of lowlands as its flow during two separate flooding episodes exceeded all records. Most reservoirs in San Diego County spilled, several for the first time since their completion. Lake Elsinore, in eastern Riverside County, caused much damage to lakeside property as it filled to an elevation not reached since 1916.

The February flooding in southern California was caused by a series of storms separated by short intervals. Some peaks of record were observed, and streamflow throughout the area remained high for a relatively long period. In many streams the volumes of sustained flow for periods of 7 and 15 consecutive days were the greatest that have occurred during the period of record.

INTRODUCTION

The storms of January-February 1980 caused significant flooding over most of California (Figure 1). The storm of mid-January covered the entire state, but most of the flooding was caused by runoff from the Sierra Nevada and the

---

Kenneth L. Wahl, John R. Crippen, and James M. Knott are with the U.S. Geological Survey in Menlo Park, California.

Note: This report is reprinted from U.S. Geological Survey Open-File Report 80-1005 with the omission of Table 6. The data in this report were collected as part of cooperative programs between the U.S. Geological Survey and various federal, state, county, and municipal agencies. The cooperation of the National Oceanic and Atmospheric Administration, the U.S. Army Corps of Engineers, and county flood control districts in southern California in furnishing unpublished precipitation, streamflow, and reservoir data is gratefully acknowledged.

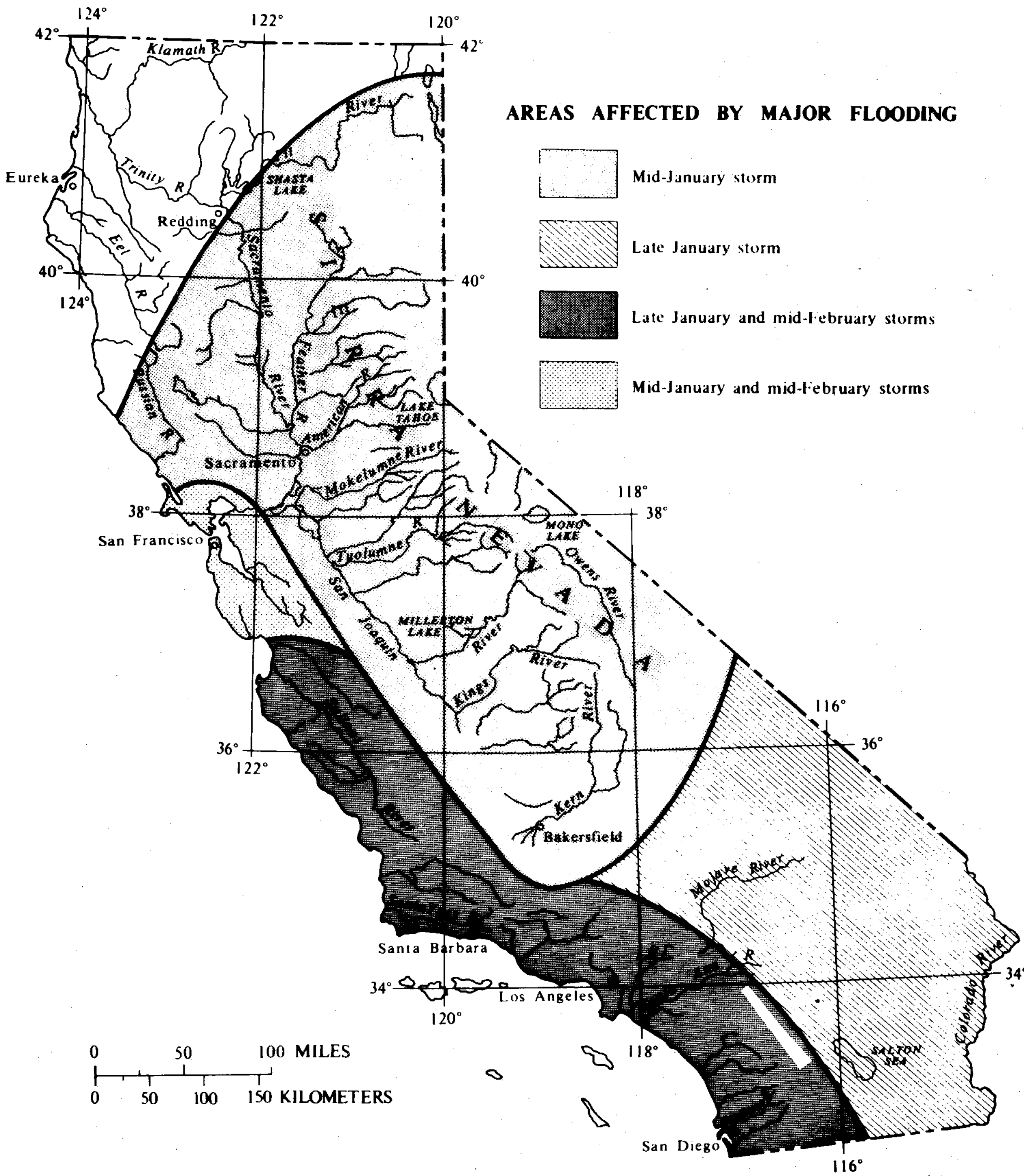


FIGURE 1 Approximate boundaries of areas in California affected by flooding in January and February 1980.

Sierra foothills; subsequent storms primarily affected southern California and coastal areas northward to San Francisco. Figure 2 shows the accumulation of precipitation during the period December 1, 1979, to April 1, 1980, at Los Angeles in the south, Yosemite Valley in the Sierra Nevada, and Shasta Dam in the north. As can be seen in Figure 2, accumulated precipitation at Shasta Dam did not exceed 120 percent of seasonal normal. In contrast, total precipitation to April 1 was 162 percent of normal at Yosemite Valley and 201 percent of normal at Los Angeles. Most of the excess occurred in mid-January at both Yosemite Valley and Los Angeles and in mid-February at Los Angeles.

The data presented in this paper are provisional, and areal coverage is not complete.

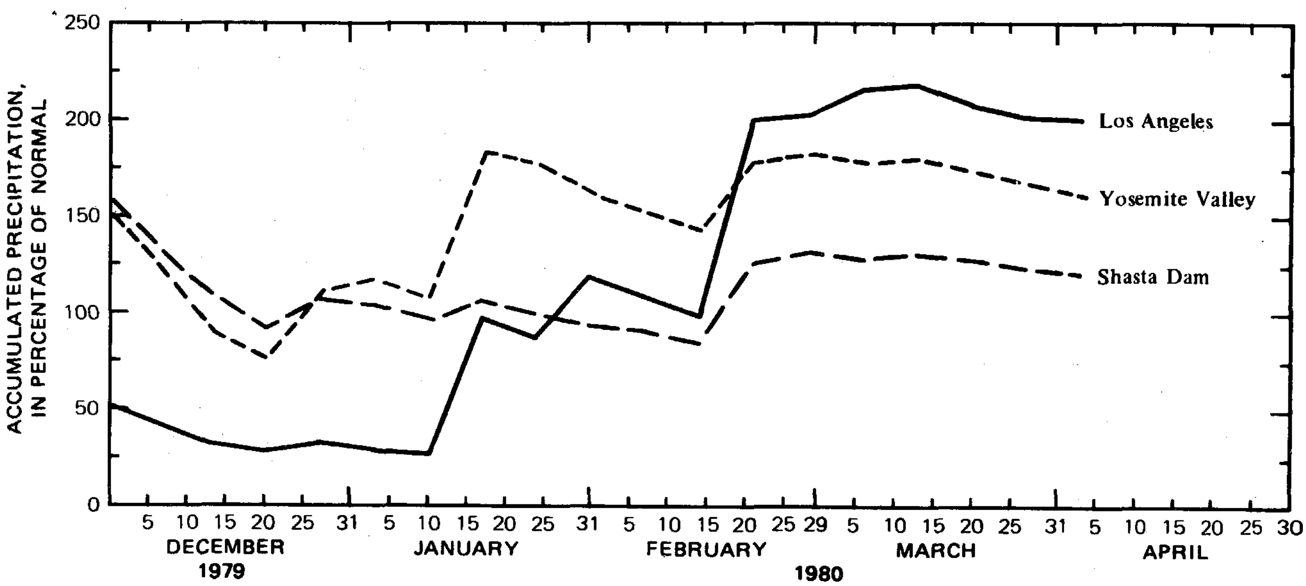


FIGURE 2 Accumulated precipitation between October 1, 1979, and indicated date, for three locations in California.

DESCRIPTION OF STORMS

Up to December 31, 1979, seasonal rainfall over California had not been excessive. The average precipitation from October 1 to December 31 at all reporting stations ranged from 127 percent of normal in the north coast drainage (see Figure 3 for definition of the reporting units) to only 22 percent of normal in the southeast desert basins (Table 1). October was wetter than normal except in the southeast desert basins, but November precipitation was below normal except in the north coast area. Precipitation

*A water year is a 12-month period ending September 30 and is designated by the calendar year in which it ends.

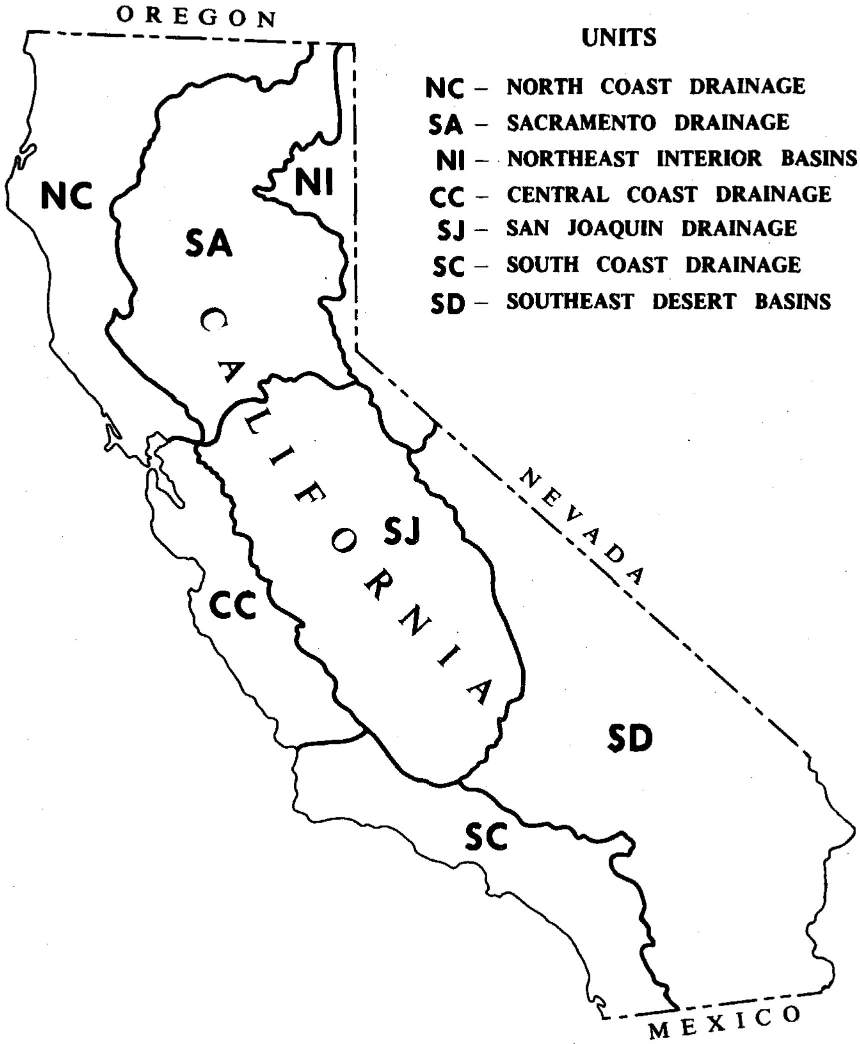


FIGURE 3 Reporting units for precipitation data.

for the month of December was also below normal despite a series of storms that affected most of the state during the period December 18-31.

Two major storms struck the state in January. The first of these occurred during January 7-19; rainfall, often heavy, was recorded at most reporting stations for the 10 consecutive days January 8-17. This storm was warm, producing rainfall in the Sierra Nevada at elevations as high as 9,000 ft. The second storm, during the period January 28-31, primarily affected the southern half of the state.

During mid-February a series of fast-moving Pacific storms brought very heavy rainfall, particularly in southern California. These storms, separated by intervals of less than 24 hours, produced heavy rainfall during February 13-22. Precipitation for that period totaled 24.26 in. at Cuyamaca and 19.79 in. at Henshaw Dam, both in San Diego County; 24.26 in. at Lake Arrowhead, San

TABLE 1 Average Accumulated Precipitation from National Oceanic and Atmospheric Administration Reporting Units

Reporting Unit	Accumulated Precipitation, 1980 Water Year					
	Oct. 1 to Dec. 31		Oct. 1 to Jan. 31		Oct. 1 to Feb. 29	
	In.	Percent of Normal	In.	Percent of Normal	In.	Percent of Normal
North coast drainage	20.92	127	28.30	114	37.60	123
Sacramento drainage	15.08	110	24.00	117	35.19	136
Northeast interior basins	7.99	106	16.60	146	23.61	167
Central coast drainage	8.76	120	14.30	123	21.43	144
San Joaquin drainage	5.86	89	14.12	139	20.62	157
South coast drainage	2.23	44	11.44	141	23.26	211
Southeast desert basins	0.51	22	3.55	98	7.81	163

Bernardino County; 12.75 in. at the Los Angeles Civic Center; and 14.25 in. at Ojai, Ventura County. These amounts range from 320 percent of normal February rainfall at Lake Arrowhead to 530 percent of normal February rainfall at Henshaw Dam. By the end of February, cumulative precipitation for most regions of California was well above normal (Table 1).

Precipitation data for January and February at selected reporting stations are summarized in Table 2.

DESCRIPTION OF FLOODS

The January and February storms produced three distinct periods of flooding, and different areas of the state were affected by each period.

Floods of Mid-January

The mid-January storm was statewide, but the most significant flooding was in the San Joaquin basin, the Sacramento basin, the central coast drainages, and the Truckee River and Honey Lake basins. Because the airmass was warm, rain fell at high elevations and melting snowpack contributed to runoff in the Sierra Nevada. Flooding was widespread in the affected areas, but flood magnitudes at most gaging stations were generally less than the historical peaks. Peak flows in the drainages of the Tuolumne, Mokelumne, Cosumnes, and American rivers, however, were among the highest in the last 20 years. On January 12 and 13 precipitation was especially heavy from Placerville, on the South Fork American River, to Cisco, on the South Yuba River. The resulting peak flows damaged several powerplants and filled small reservoirs in the Sierra Nevada with debris. Runoff in the area equaled or exceeded the

TABLE 2 Precipitation at Selected Locations in California During January and February 1980 (in.)

				January				February		
Precipitation Station	Altitude (ft)	January 7-19 Total	January 28-31 Total	Maximum one Day	Total for Month	Departure from Normal	February 13-22 Total	Maximum one Day	Total for Month	Departure from Normal
North coast drainage										
Eureka	60	2.92	T	0.75	3.19	-4.23	2.69	1.13	4.67	-0.48
Healdsburg	102	8.77	T	2.50	8.82	-1.04	12.50	3.20	14.61	7.89
Sacramento drainage										
Red Bluff	342	2.77	0	.92	2.84	-1.64	6.45	1.72	7.77	4.60
Placerville	1890	13.39	0.60	3.84	15.33	7.50	9.62	1.80	11.51	5.91
Sacramento FAA	18	5.62	0	1.23	5.64	1.91	6.41	1.38	7.12	4.44
Northeast interior basins										
Tahoe City	6230	13.46	0.19	3.49	14.89	8.11	10.47	2.12	11.07	6.48
Central coast drainage										
Mount Hamilton	4206	6.42	0	1.21	6.42	1.96	3.93	.92	4.28	.23
San Luis Obispo Poly	315	8.47	0.53	2.60	9.52	4.92	11.47	3.98	11.91	7.89
Santa Cruz	130	9.92	0	4.14	9.97	3.24	7.87	1.55	8.69	3.43
San Joaquin drainage										
Fresno	328	3.78	0.04	.96	3.83	1.99	3.18	1.57	3.30	1.58
Hetch Hetchy	3870	14.67	0.71	3.07	16.27	10.42	11.52	*	12.73	7.92
Yosemite Park HDQ	3966	15.59	0.29	4.03	16.54	10.03	13.53	3.07	14.24	8.71
South coast drainage										
Cuyamaca	4640	13.14	9.23	4.40	22.37	16.78	24.34	5.35	24.34	18.93
San Diego	13	2.96	2.53	1.92	5.58	3.70	4.47	1.41	4.47	2.99
Escondido	660	6.08	5.41	3.24	11.49	*	10.11	1.96	10.11	7.90
Henshaw Dam	2700	10.63	8.14	5.60	18.77	14.54	19.79	3.85	19.79	16.06
Palomar Mt. Observatory	5545	11.27	7.36	5.65	18.63	13.78	19.89	2.90	19.89	15.24
Laguna Beach	35	4.68	2.93	2.25	7.61	5.33	9.64	1.70	9.64	6.37
Riverside Fire Station 3	840	3.34	2.13	1.47	5.47	3.66	6.31	1.27	6.31	4.56
Los Angeles Civic Center	257	4.66	2.84	2.44	7.50	4.50	12.75	3.03	12.75	9.98
Ojai	750	6.70	2.11	2.15	8.81	4.18	14.25	5.60	14.25	10.08
Santa Barbara	5	5.71	1.00	1.94	6.71	2.77	8.98	3.48	8.98	6.53
Southeast desert basins										
Lake Arrowhead	5205	14.14	6.68	6.26	22.15	14.01	24.26	4.55	24.26	16.64
Palm Springs	425	1.52	2.62	2.02	4.14	3.01	5.41	1.14	5.41	4.63

Source: National Oceanic and Atmospheric Administration.

Note: Dates shown refer to those in Climatological Data reports of the National Oceanic and Atmospheric Administration, National Climatic Center. T = trace; * = not determined.

December 1964 flood. At North Fork of Middle Fork American River near Foresthill and at Maine Bar Canyon Creek near Greenwood, runoff was 339 and 346 cu ft/s-sq mi, respectively.

The flows of most rivers draining the Sierra Nevada are regulated by reservoirs located upstream from the lowlands of the Central Valley. Normal operating procedures for these reservoirs maintain a storage capacity to receive the high runoff expected in the spring and early summer. Runoff from the mid-January storms encroached on this flood storage space in 15 of the 16 major reservoirs, causing some anxiety concerning difficulties that might arise if later storms and snowmelt should produce excessive rates of inflow.

The unusually high discharges of the Sacramento and San Joaquin rivers coincided with abnormally high tides and winds. This combination of stresses caused levees to fail on both the Holland and Webb tracts, and the 9-mile-long lake that was formed in the Sacramento-San Joaquin Delta flooded about 10,000 acres of prime agricultural land. One person was drowned and about 900 head of cattle were lost when the levees failed.

Rainfall totals in southern California had been well below normal prior to the mid-January storm. Consequently, runoff from this storm was not extreme. The replenishment of soil moisture, however, set the stage for flooding from the storms that were to follow later in January and February.

Floods of Late January

The storm of January 28-31 brought large amounts of rainfall to the south coastal and southeast desert areas but only light precipitation to other areas of the state. Cuyamaca and Henshaw Dam in San Diego County reported three-day totals of 9.23 in. and 8.14 in., respectively, and Lake Arrowhead reported a one-day rainfall of 6.26 in. on January 28. Most peak flows in the area were well below the historical record peaks. For example, the January 29, 1980, peak at Santa Ana River at E Street, near San Bernardino (station 11059300), was 22,000 cu ft/s, below the 1969 peak discharge of 28,000 cu ft/s. However, the peak discharge of 3,550 cu ft/s at the gaging station on East Twin Creek near Arrowhead Springs (station 11058500) was the highest for the period of record, dating back to 1919. Farther south, in the Tijuana River basin, heavy runoff from the Rio Las Palmas into Rodriguez Reservoir in Mexico caused concern for the safety of the dam and necessitated large releases. These releases reached 28,000 cu ft/s on January 30 and combined with the floodwaters from the Tijuana River to produce an estimated peak discharge of 32,000 cu ft/s at the Tijuana River near Nestor (station 11013500). The previous record peak discharge at the Tijuana River gage was 17,700 cu ft/s in 1937. The January peak produced widespread flooding along the Tijuana River downstream from the levees that end at Dairy Mart Road, about 2 miles downstream from the international boundary. Flooding was to occur again in mid-February.

Floods of Mid-February

Little rain fell in California in early February, as the southern part of the state began the task of cleaning up from the late January storms. Then during February 13-22, a series of storms swept through the south and central coastal areas, bringing record amounts of precipitation and runoff that caused damage to roads and property. By the time the storms had ended, eight counties had been declared federal disaster areas and 18 lives had been lost as a result of the storms. This series of storms, like that at the end of January, struck hardest in southern California and Baja California; however, it also produced significant flooding to the north in the San Francisco Bay area and in the Salinas River basin.

Flooding was only one of the problems caused by storms. High winds and wave action caused heavy damage in several coastal areas; mudflows and slope failures due to saturated soils caused extensive property damage. Broken sewer lines caused contamination of beaches.

Coastal Basins South of the Santa Ana River

San Diego County again was hard hit, with extensive flooding on the Tijuana River and in the Mission Valley area along the lower San Diego River. As in late January, heavy runoff from Baja California and concern for the safety of Rodriguez Dam necessitated large releases from the reservoir. These releases, although not as great as those of January 30, combined with floodflow from the Tijuana River to produce an estimated peak discharge on the Tijuana River near Nestor on February 21 of 34,200 cu ft/s, slightly larger than the previous record peak of January 30. Flooding was extensive downstream from San Ysidro, and the bridge on Hollister Road was destroyed. Figure 4 is the hydrograph of daily discharge on the Tijuana River near Nestor at the international boundary.

Except for Lake Henshaw, all major reservoirs in San Diego County spilled as a result of the storm. Lower Otay Reservoir, however, spilled only 350 cu ft/s, and that did not occur until March 11. Estimated inflow and outflow data for selected reservoirs in San Diego County are summarized in Table 3.

Peak discharges at gaging stations in the San Luis Rey River and Santa Margarita River basins were generally the highest in the last 50 years, approaching the magnitudes of the 1927 floods. Vail Lake on the Temecula River spilled in February for the first time since the dam was completed in 1948. The daily discharge hydrograph for Murrieta Creek at Temecula (station 11043000) is shown in Figure 5.

Coastal Basins from the Santa Ana River to the Los Angeles River

Peak discharges in the Santa Ana River basin were not as high as they were in either 1969 or 1938. Runoff volumes, however, were among the highest of this century and are discussed in a later section. Storage in the Prado Flood Control Reservoir reached a maximum of about 111,000 acre-ft on February 22, the second highest of record. The highest storage of record, 130,000 acre-ft, occurred on February 25, 1969. Figure 6 shows storage as a function of time for January through March.

Flooding from Lake Elsinore damaged many homes and facilities in low-lying areas. Skylark Airport, at the southeast end of the lake, was inundated as

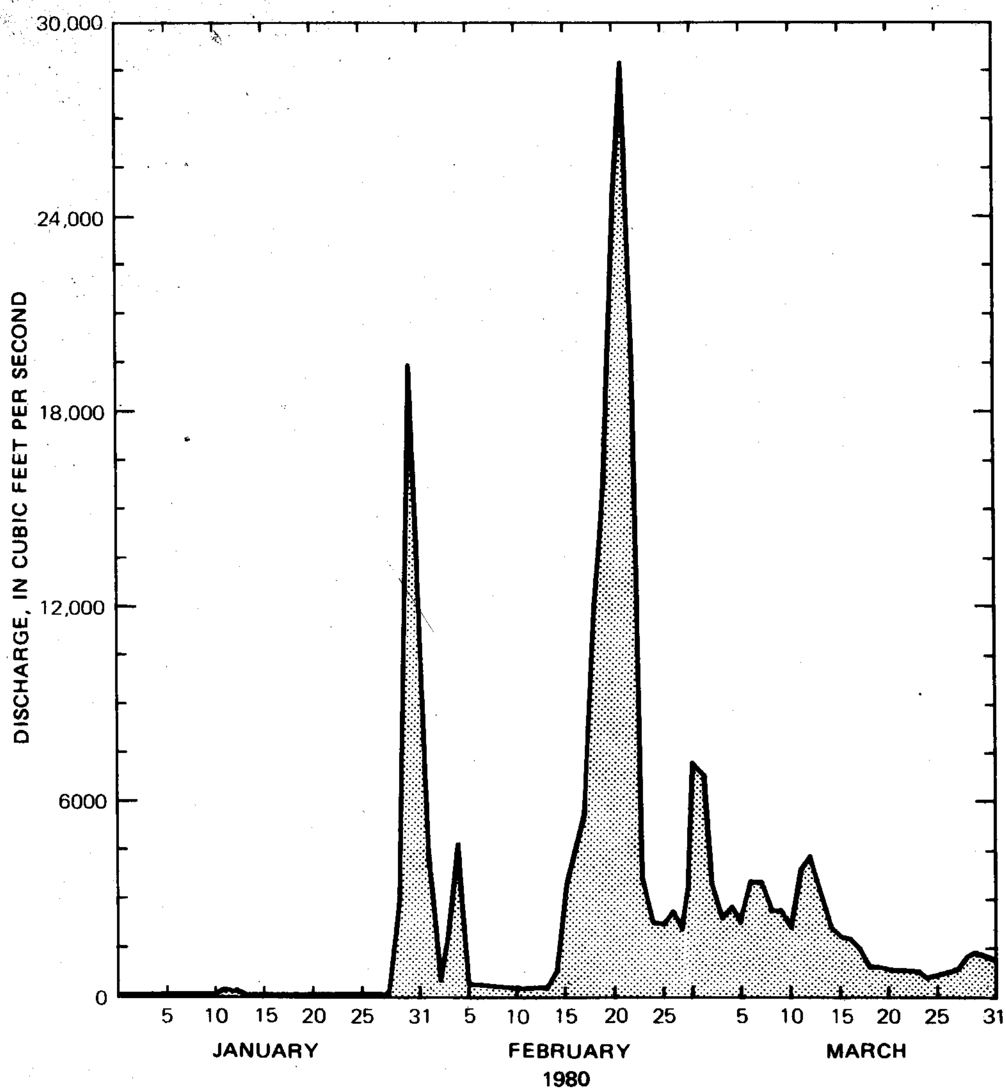


FIGURE 4 Daily discharge for Tijuana River near Nestor.

the surface area of the lake spread to much more than its normal size. Inflow to Elsinore is from the San Jacinto River, with slight additional contributions from small tributary basins. The daily discharge hydrograph for the San Jacinto River near Elsinore (station 11070500) is shown in Figure 7. Historically, the lake is intermittent, with the lake bed remaining dry for many years in succession. Then, during wet periods, it becomes covered to shallow depths for as much as several square miles. The natural outlet of the lake is Temescal Creek. There probably was outflow down Temescal Creek in 1862, and outflow is known to have occurred in 1872, 1883-84, and 1916. The lake bed was dry in the 1960s until 1965, when Colorado River water was brought in via the San Jacinto River. Since that time a lake of about 6 square miles in area has been maintained.

On February 13, 1980, the lake surface was recorded by the U.S. Army Corps of Engineers to be at 1246.59 ft, gage datum, and its contents was 61,200

TABLE 3 Estimated Peak Inflow and Outflow from Selected Reservoirs in San Diego County

River Basin	Reservoir	Inflow (cu ft/s)	Outflow (cu ft/s)	Date of Peak Outflow
Tijuana	Barrett	a	8,000	February 21
	Morena	a	2,900	February 21
	Rodriguez[a]	a	28,000	January 30
Sweetwater	Loveland	a	5,000	February 21
	Sweetwater	a	7,000	February 21
San Diego	El Capitan	40,000	1,080	February 24
	San Vicente	11,500	6,000	February 21
San Dieguito	Sutherland	a	6,100	February 21
	Lake Hodges	28,000	22,000	February 21

Note: a = no estimate.

[a]Located in Mexico.

Source: County of San Diego, Department of Sanitation and Flood Control.

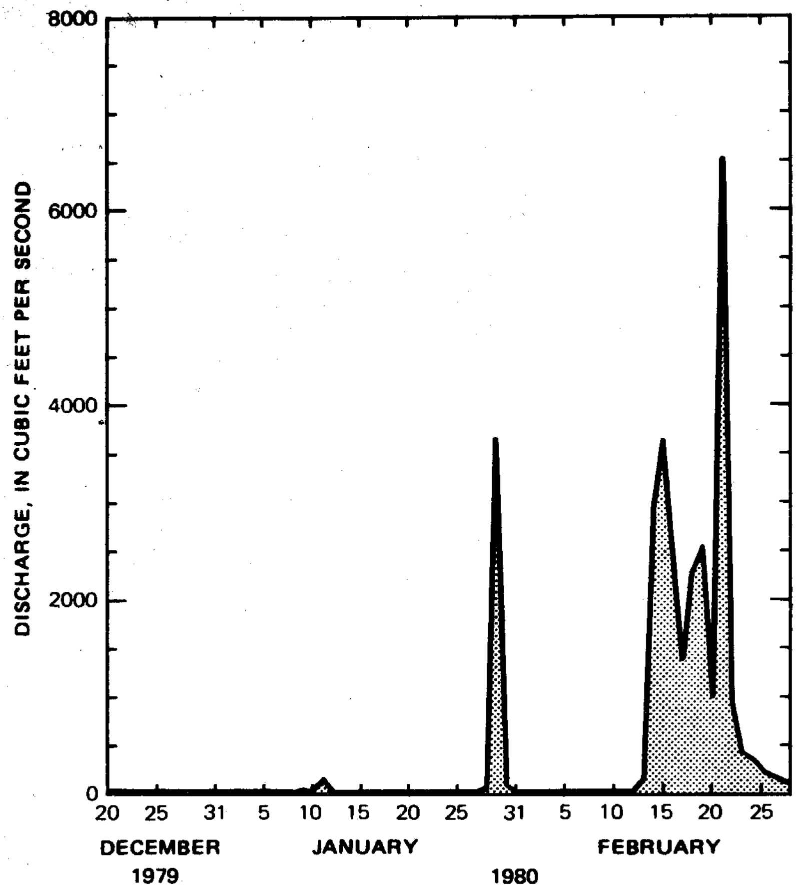


FIGURE 5 Daily discharge for Murietta Creek at Temecula.

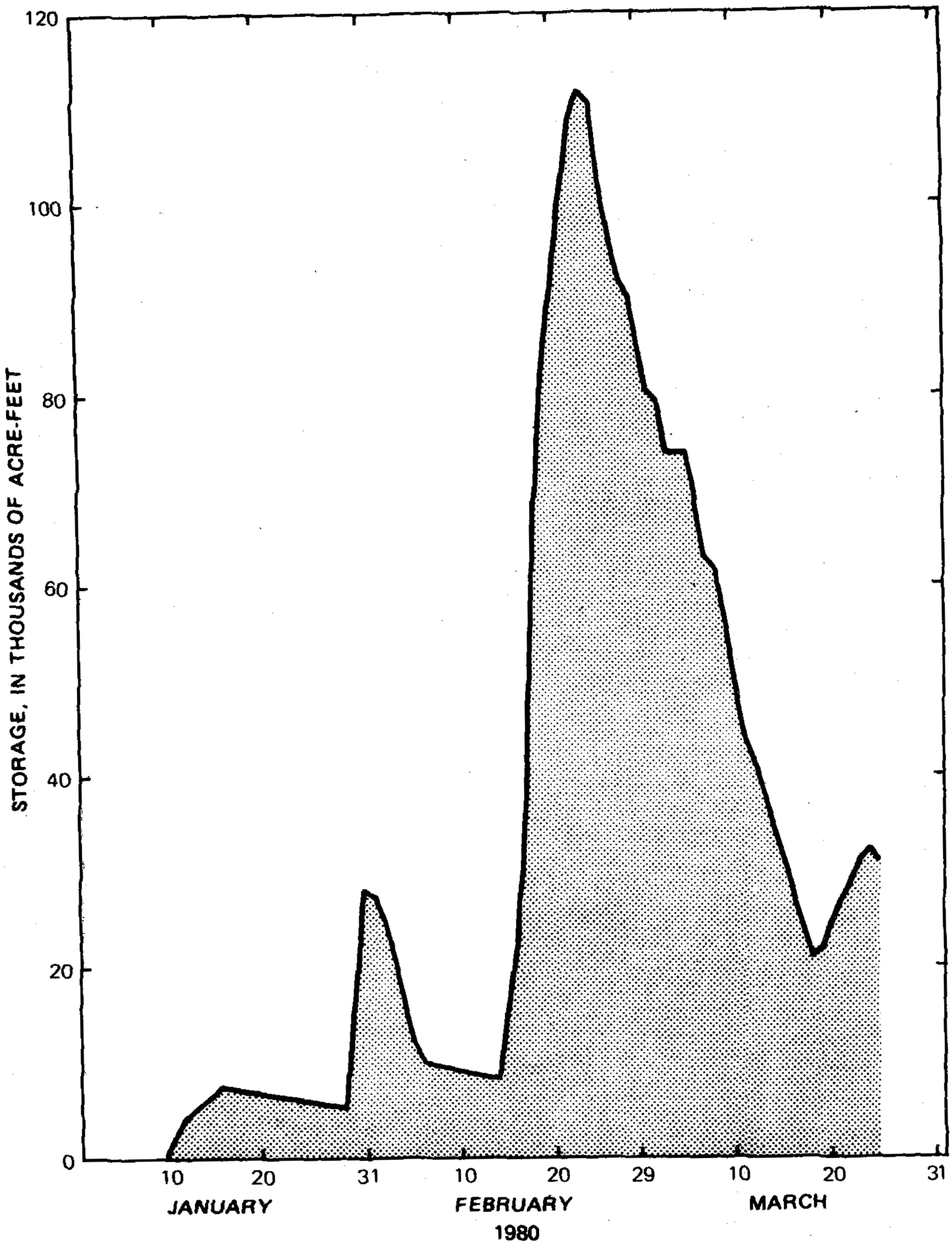


FIGURE 6 Storage in Prado Flood Control Reservoir during January-March 1980.

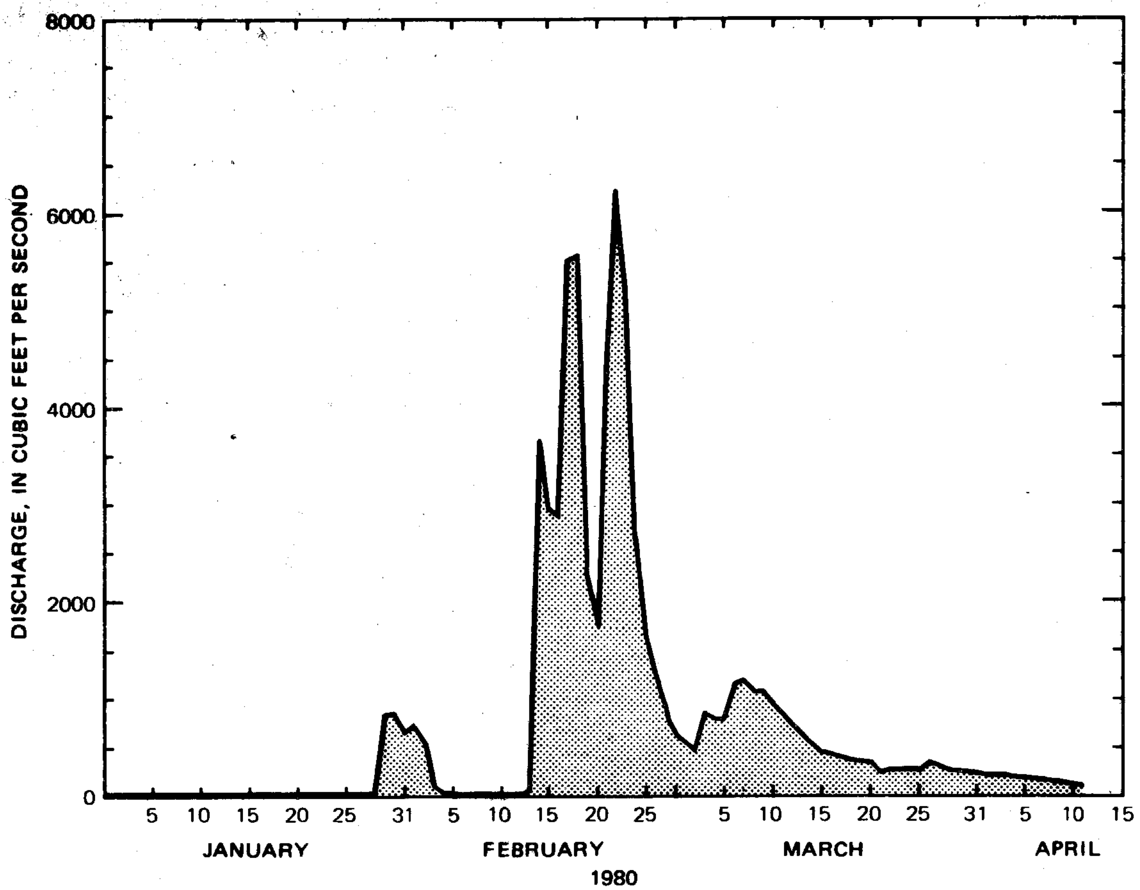


FIGURE 7 Daily discharge for San Jacinto River near Elsinore.

acre-ft. Inflow reached a maximum of slightly more than 5,000 cu ft/s on February 22 and then decreased, except for a slight rise after rainfall in early March, to less than 100 cu ft/s in mid-April. After clearing and repair of the outlet channel, outflow started on March 8 and reached a maximum rate of almost 240 cu ft/s later in the month. The stage of Lake Elsinore reached a maximum on March 20-21 of 1265.72 ft; the corresponding volume of the lake was 163,400 acre-ft and its surface area was about 10 square miles. Data from the Corps indicate that inflow from February 13 to March 21 was 107,000 acre-ft, with an additional inflow of 5,800 acre-ft by April 11. Figure 8 shows the changes in stage and contents of the lake from February 1 to April 11.

Flooding in the headwater tributaries of the San Gabriel and Los Angeles rivers was comparable with the extreme floods of 1969. On the main stem of the San Gabriel River, however, flood control reservoirs reduced peak discharge to below the 1969 magnitudes. By contrast, the February 16 peak discharge for the Los Angeles River at Long Beach (station 11103000) was 125,000 cu ft/s, the highest at that site since records began in 1928. The hydrograph of daily discharge for Arroyo Seco near Pasadena (station 11098000), tributary to the Los Angeles River, is shown in Figure 9.

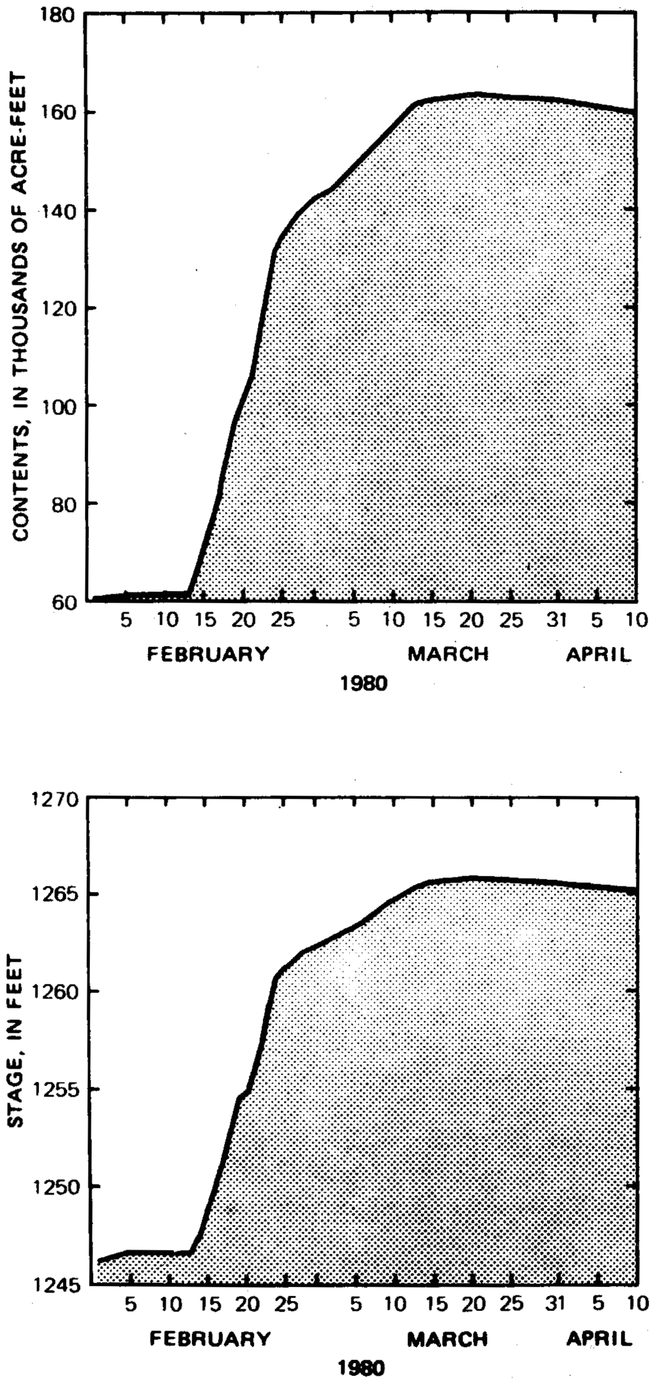


FIGURE 8 Changes in stage and contents of Lake Elsinore.

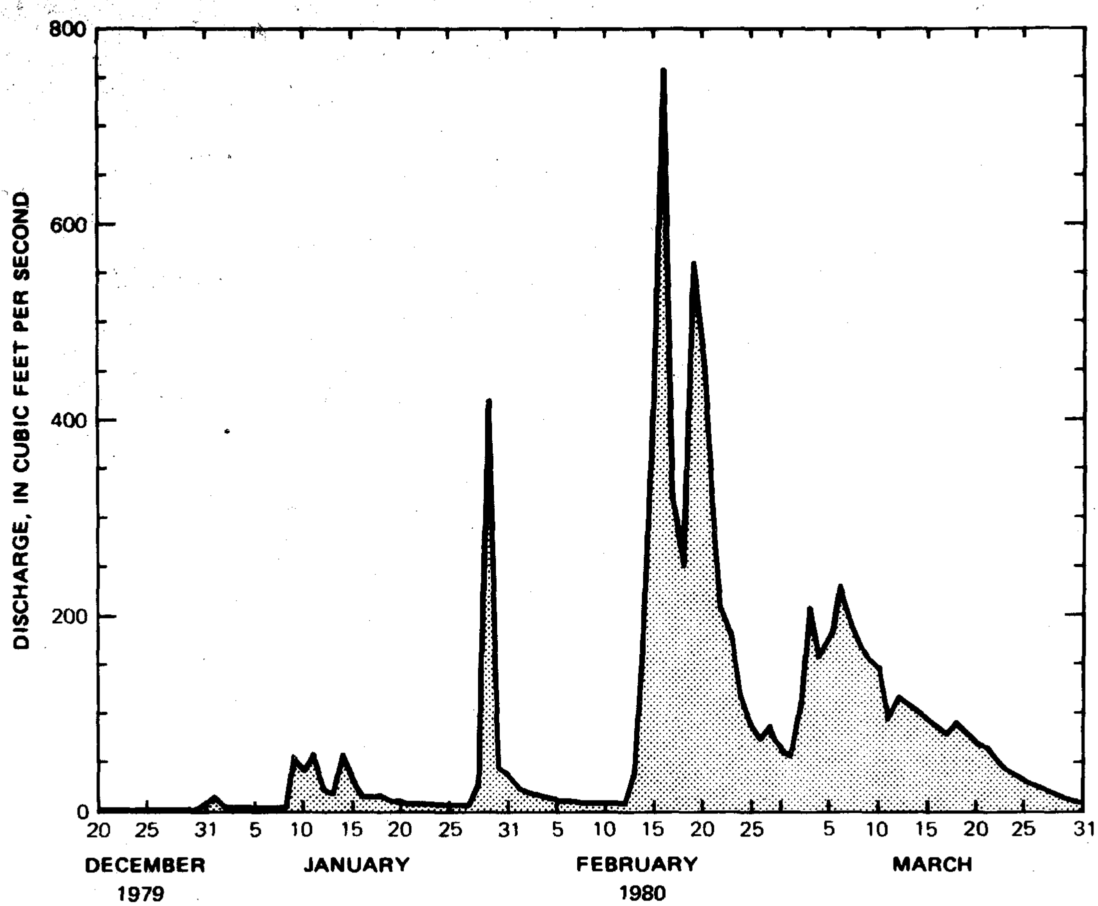


FIGURE 9 Daily discharge for Arroyo Seco near Pasadena.

Coastal Basins North and West of Los Angeles

Flood damage was extensive in the small basins between the Los Angeles River and the Santa Clara River. Homes were damaged by mudflows and floodwaters in the Topanga Creek and Malibu Creek basins. Raw sewage flowed down Malibu Creek after a sewer line was broken by floodwaters; the resulting contamination caused health officials to close about 65 miles of beaches for several weeks to swimmers and surfers. Parts of the Point Mugu U.S. Naval Air Missile Test Center were flooded when a dike along Calleguas Creek failed.

Flooding in the Santa Clara River basin and in Santa Barbara County was generally less severe than the record floods in 1969 and 1978. Daily discharge hydrographs for Sespe Creek near Fillmore (station 111130000) and Santa Clara River at Montalvo (station 11114000) are shown in Figures 10 and 11.

In the area extending north from San Luis Obispo County to the San Francisco Bay area, peak flows of many small streams were among the highest in 20 years. The 1980 peaks in the counties surrounding San Francisco Bay rivaled, but usually did not exceed, peaks in the 1955 and 1958 floods.

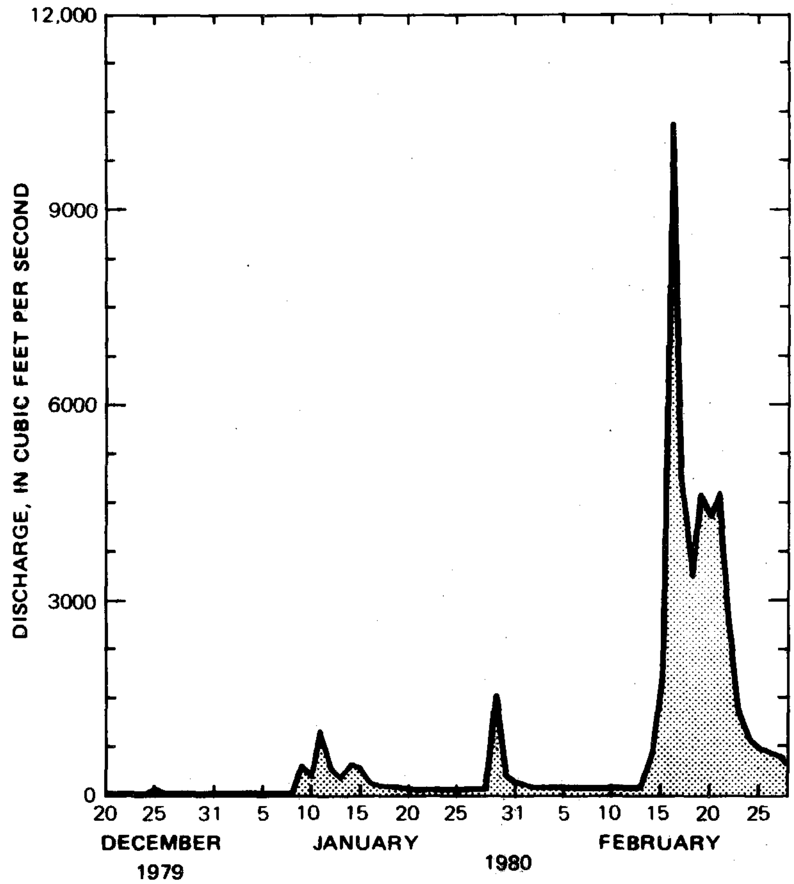


FIGURE 10 Daily discharge for Sespe Creek near Fillmore.

Significance of Floods

The individual storms of January-February 1980 followed a pattern not unusual in California. However, the number of storms and the short intervals between them during February 13-22 were unusual for southern California. Their closeness in time ensured that each succeeding rainfall would strike an area already primed to yield substantial runoff. Few of the storms alone would have caused major flooding; however, the rapid sequence of storms resulted in extreme volumes of flows and in flooding that was unusually high and destructive. Table 4 shows for selected sites, as indexes of flood volume, the highest average flows for periods of 7 consecutive days and 15 consecutive days in 1980; their ranking in order of magnitude when compared with similar flow durations during the period of record; and the previous highs of such flows. Many streams south of the Los Angeles basin carried the highest 7- and 15-day volumes yet recorded. Streams to the north, although unusually high, carried volumes substantially less than the previous maximums for 7 and 15 days.

In southern California sustained high flow constitutes an important source of recharge to the groundwater basins. Because of the seasonal concentration

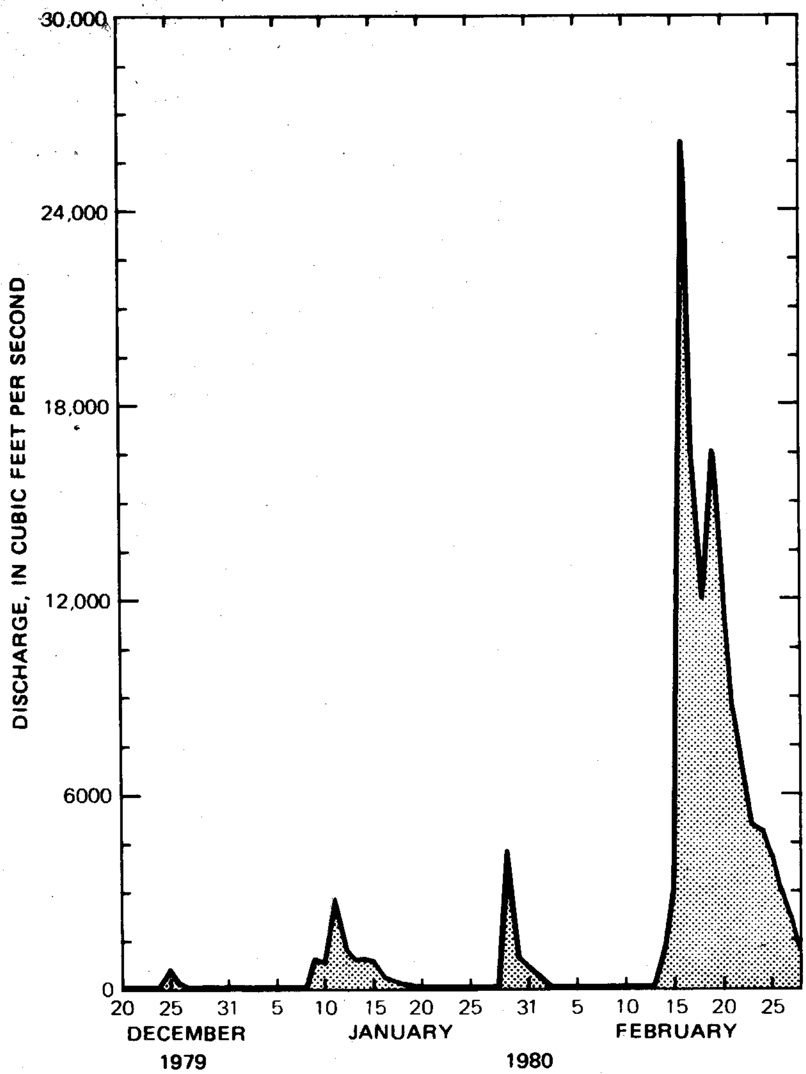


FIGURE 11 Daily discharge for Santa Clara River at Montalvo.

of precipitation during the winter months, followed by pumping during the summer, groundwater levels tend to show large seasonal fluctuation, rising in the winter and early spring and falling in summer and autumn. In addition to this seasonal cycle, recharge varies greatly from year to year as a result of the large variance in annual precipitation. Figure 12 shows changes in the water level at an index well in Baldwin Park, about 15 miles east of central Los Angeles, from January 1977 to late May 1980.

DAMAGE ESTIMATES

Eight counties, including all of southern California except Imperial County, were declared disaster areas. They are Los Angeles, Orange, Santa Barbara, San Bernardino, Riverside, San Diego, Ventura, and, farther north, Santa Cruz (Figure 13). Eighteen lives were lost in these counties as a result of the January and February storms and floods.

TABLE 4 Sustained Floodflows at Selected Sites During Floods of 1980 in Southern California

Station No.	Name	Period of Record	High 7 days				High 15 days			
			1980		Previous high		1980		Previous high	
			Flow	Rank	Flow	Year	Flow	Rank	Flow	Year
11012500	Campo Creek near Campo	1937-80	219	1	88	1941	149	1	67	1941
11013500	Tijuana River near Nestor	1937-80	15,700	1	5,670	1941	9,330	1	4,250	1941
11015000	Sweetwater River near Descanso	1907-80[a]	1,110	2	1,260	1916	602	2	1,040	1916
11043000	Murrietta Creek at Temecula	1931-80	2,800	1	2,170	1969	1,670	1	1,030	1969
11070500	San Jacinto River near Elsinore	1917-80	4,410	2	4,490	1927	3,180	1	2,360	1927
11074000	Santa Ana River below Prado Dam	1941-80	5,910	1	5,320	1969	4,750	1	3,580	1969
11098000	Arroyo Seco near Pasadena	1914-80	440	6	1,230	1914	272	8	639	1914
11113000	Sespe Creek near Fillmore	1928-80	4,950	7	11,500	1969	2,780	8	7,220	1969
11114000	Santa Clara River at Montalvo	1950-80	14,100	3	25,400	1969	8,280	3	13,700	1969
11118500	Ventura River near Ventura	1930-80	4,740	4	6,970	1969	2,640	5	3,960	1969
11132500	Salsipuedes Creek near Lompoc	1942-80	526	3	925	1978	272	4	523	1962
11140000	Sisquoc River near Garey	1942-80	1,800	6	6,250	1969	1,080	5	3,780	1969

Note: Average flows for highest 7 and 15 consecutive days. All flows in cubic feet per second. To compute acre-feet, multiply 7-day flow by 9.917 and 15-day flow by 29.752. Rank of 1 indicates highest event during period of record; 2 indicates second highest; and so forth. All the sustained flows shown for 1980 began during the period February 13-19.

[a]No record for 1928-56.

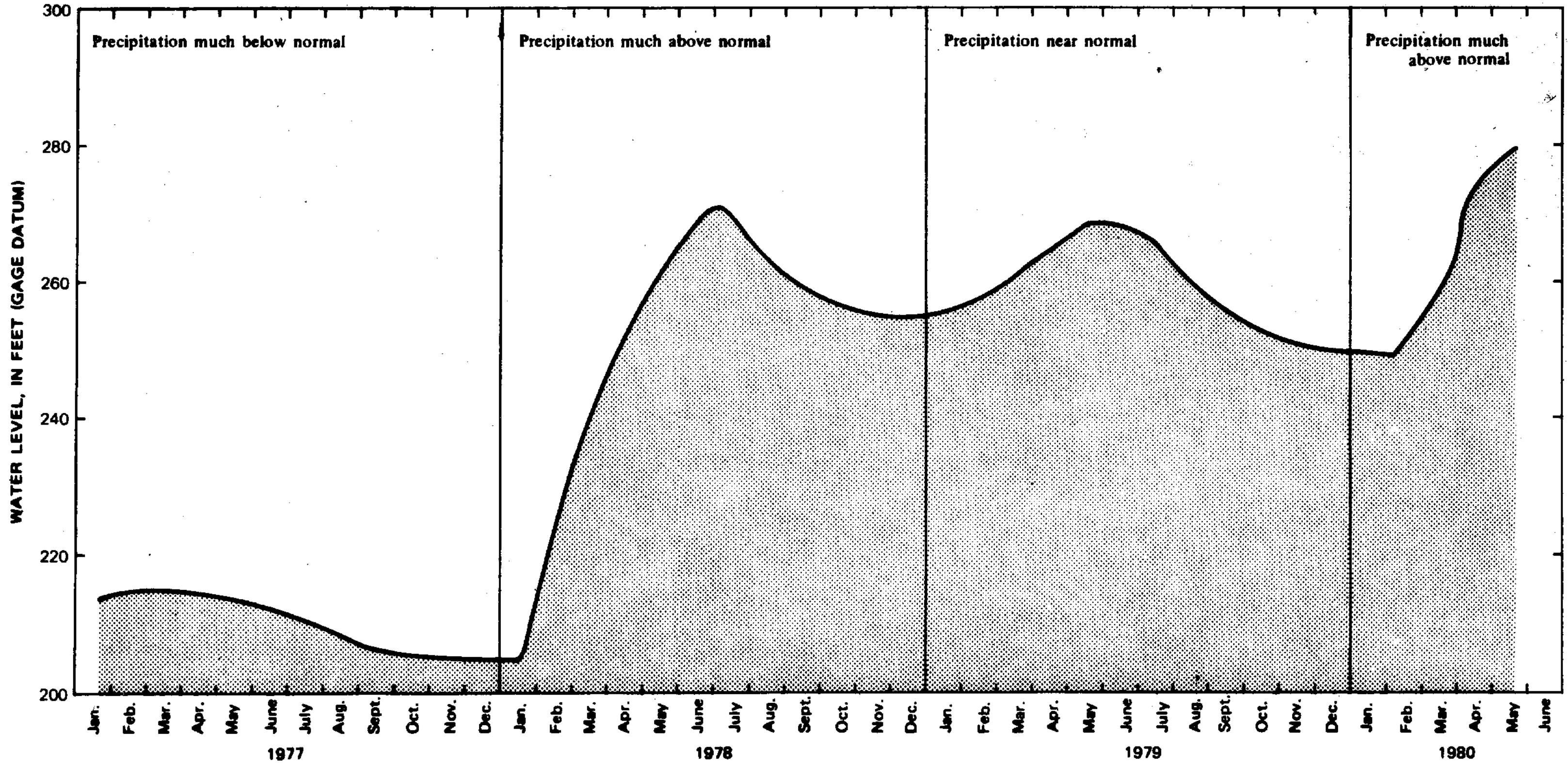
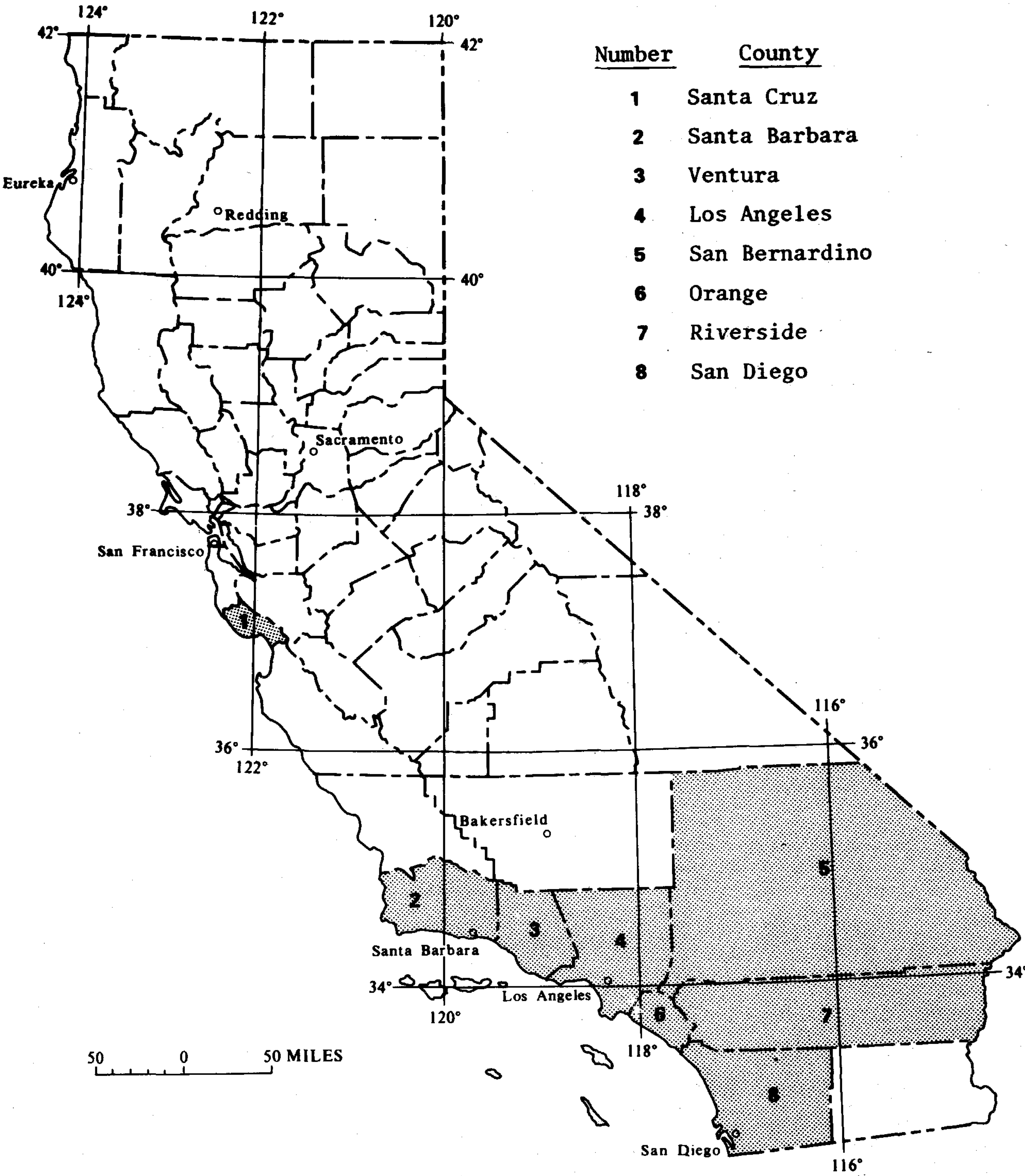


FIGURE 12 Changes of groundwater level in Baldwin Park well (1S/10W-7R2).



Number	County
1	Santa Cruz
2	Santa Barbara
3	Ventura
4	Los Angeles
5	San Bernardino
6	Orange
7	Riverside
8	San Diego

FIGURE 13 Counties declared disaster areas as a result of the January-February 1980 floods.

Preliminary flood damage estimates for the eight-county area were coordinated by the Federal Emergency Management Agency (oral communication, 1980). Total damages were estimated to be almost $350 million. The breakdown of these estimates by types of property damaged is shown below:

Type of Property	Damage (millions of dollars)
Public facilities	175
Private property	94.8
Business	30
Agriculture	48.6
Total	348.4

SEDIMENT DATA

Selected sediment samples, obtained during the floods of January and February 1980 in southern California, were analyzed for sediment concentration, and the data are presented in Table 5. The limited time available for analysis of samples and of streamflow data precluded the detailed computation of sediment discharge for individual flood periods. Water and sediment discharge data presented here are provisional; final data will be published in the annual series Water Resources Data for California.

Data on water and sediment discharge during the 1980 floods from three sites (Santa Ana River at Santa Ana, Santa Clara River at Montalvo, and Ventura River near Ventura) are plotted in Figures 14-16. Similar data obtained during the floods of 1969 and 1978 are also plotted for comparison. Analysis of sediment samples is not yet complete enough to warrant estimates of the total sediment transported during the 1980 floods.

TABLE 5 Suspended-Sediment Data for Selected Gaging Stations in Southern California During January-February 1980

Date	Time (hours)	Gage Height (ft)	Discharge (cu ft/s)	Suspended Sediment Concentration (mg/liter)	Suspended Sediment Tons Per Day
11046550 San Juan Creek at San Juan Capistrano					
January 10	1200	11.73	17	159	7.3
January 14	1055	12.08	17	101	4.6
January 17	1335	12.01	12	281	9.1
January 18	1420	-	12	72	2.3
February 17	1240	-	3,000	7,390	59,900
February 18	1010	-	9,560	29,700	767,000
11048500 San Diego Creek at Sand Canyon Avenue, Near Irvine					
January 10	1455	11.83	8.7	446	10
January 11	1100	15.40	1,700	10,200	46,800
January 11	1225	13.22	240	6,900	4,470
January 14	1540	-	10	566	15
January 15	1450	10.72	7.0	192	3.6
January 17	1115	10.71	5.3	105	1.5
January 18	1105	10.91	15	556	23
January 28	1345	11.85	220	10,200	6,060
January 29	1030	10.86	100	8,820	2,380
January 30	1430	-	17	924	42
January 31	1415	-	15	925	37
February 17	0945	9.95	99	4,450	1,190
February 18	1200	10.37	259	17,000	11,900
February 18	1240	10.18	313	11,600	9,800
11074000 Santa Ana River Below Prado Dam					
January 2	1600	2.86	203	83	45
January 16	1040	3.94	705	180	343
January 23	1400	2.93	226	42	26
February 4	0920	5.03	1,950	128	674
February 4	1210	5.17	2,200	127	754
February 4	1500	5.17	2,200	134	796
February 5	0915	5.15	2,090	106	598
February 5	1440	4.85	1,620	126	551
February 6	0815	3.04	250	110	74
February 7	1030	3.03	246	91	60

TABLE 5 Suspended-Sediment Data for Selected Gaging Stations in Southern Califonria During January-February 1980 (Cont)

Date	Time (hours)	Gage Height (ft)	Discharge (cu ft/s)	Suspended Sediment Concentration (mg/liter)	Suspended Sediment Tons Per Day
February 8	1330	3.01	314	49	42
February 16	0930	5.38	2,600	157	1,100
February 16	1520	4.72	1,500	162	656
February 17	0830	5.44	2,800	1,890	14,300
February 17	1430	5.56	3,000	720	5,830
February 17	1630	5.55	2,990	488	3,940
February 18	0815	6.04	4,300	437	5,070
	11075755 Santa Ana River at Ball Road, in Anaheim				
January 14	1400	1.77	114	538	166
January 15	1245	1.84	186	610	306
January 17	1330	1.96	324	483	423
January 17	1525	1.92	296	478	382
January 18	1345	1.96	324	451	395
January 29	1425	2.46	859	2,950	6,840
January 29	1700	2.35	708	4,510	8,620
January 30	0945	2.87	1,630	5,770	25,400
January 31	1315	2.85	1,580	2,200	9,390
February 5	1700	2.85	1,580	1,180	5,030
February 7	1045	1.78	211	158	90
February 8	1445	1.61	132	122	43
February 13	1700	2.79	1,450	6,190	24,200
February 14	1045	2.60	1,080	5,760	16,800
February 14	1645	2.57	1,030	2,920	8,120
February 16	1430	3.74	4,770	5,860	75,500
February 17	0900	3.60	4,100	4,310	47,700
February 19	1330	4.10	6,900	6,320	118,000
	11078000 Santa Ana River at Santa Ana				
January 10	1030	5.30	17	171	7.8
January 11	1600	6.47	340	470	431
January 14	1330	5.78	140	245	93
January 15	1400	5.97	177	301	144
January 16	1445	6.22	276	196	146
January 17	1145	6.25	297	264	212
January 22	1550	5.61	96	113	29

TABLE 5 Suspended-Sediment Data for Selected Gaging Stations in Southern California During January-February 1980 (Cont)

Date	Time (hours)	Gage Height (ft)	Discharge (cu ft/s)	Suspended Sediment Concentration (mg/liter)	Suspended Sediment Tons Per Day
January 29	1300	7.17	1,600	5,230	22,600
January 30	1140	7.57	2,300	3,110	19,300
January 31	1100	-	1,500	1,410	5,710
February 5	1625	-	1,000	765	2,070
February 6	1445	-	977	406	1,070
February 7	1300	-	168	139	63
February 8	1700	-	60	114	18
February 11	1445	-	35	41	3.9
February 13	1500	-	3,400	2,790	25,600
February 14	1315	-	1,100	4,430	13,200
February 14	1630	-	950	2,200	5,640
February 15	1555	-	2,210	4,200	25,100
February 16	1400	-	8,300	5,990	134,600
February 16	1715	-	3,600	4,860	47,200
February 17	1250	-	3,160	3,970	33,900
February 17	1500	-	3,500	4,420	41,800
February 19	1500	-	7,000	7,300	138,000
	11114000 Santa Clara River at Montalvo				
January 9	0900	2.05	221	3,840	2,290
January 9	1600	4.00	2,640	4,650	33,100
January 10	1030	2.56	538	1,450	2,110
January 10	1530	2.60	570	2,420	3,720
January 11	0900	4.40	3,630	18,500	181,000
January 11	1415	4.23	3,180	10,300	88,400
January 12	1030	3.30	1,360	1,720	6,300
January 12	1600	3.23	1,260	832	2,830
January 13	0900	2.85	804	456	990
January 13	1600	2.73	685	1,720	3,180
January 14	1915	2.86	814	1,860	4,090
January 14	1320	3.03	1,010	2,830	7,720
January 17	1000	2.13	259	466	326
January 25	0915	1.47	47	34	4.3
January 28	1600	1.46	45	4,280	520
January 29	0930	5.61	8,400	32,000	726,000
January 30	1100	3.10	868	1,060	2,480
January 31	1000	2.87	640	376	650

TABLE 5 Suspended-Sediment Data for Selected Gaging Stations in Southern California During January-February 1980 (Cont)

Date	Time (hours)	Gage Height (ft)	Discharge (cu ft/s)	Suspended Sediment Concentration (mg/liter)	Suspended Sediment Tons Per Day
February 11	1600	1.98	2.4	39	.25
February 13	1100	1.98	2.4	3,230	21
February 13	1600	1.99	2.6	3,930	28
February 14	1030	3.65	1,940	13,400	70,200
February 15	1030	3.80	2,220	3,830	23,000
February 16	1000	4.59	5,890	8,340	133,000
February 16	1400	7.60	29,500	9,360	746,000
February 17	1130	5.58	10,900	23,900	703,000
February 18	0945	5.70	11,600	22,700	711,000
February 19	1000	6.45	17,300	29,600	1,380,000
February 19	1500	7.67	29,500	17,800	1,420,000
February 20	0815	5.97	10,500	15,300	434,000
February 20	1120	5.79	9,390	12,000	304,000
February 20	1600	6.43	13,400	14,600	528,000
February 21	0830	5.47	7,690	13,200	274,000
February 22	0830	5.53	7,990	18,700	403,000
February 23	1030	4.97	5,510	8,240	123,000
February 24	0900	4.87	5,130	9,650	134,000
February 25	0900	-	4,100	2,080	23,000
	11118500 Ventura River Near Ventura				
January 9	0850	3.32	43	78	9.1
January 9	0930	3.35	48	101	13
January 9	1310	3.47	67	303	55
January 9	1635	3.31	42	59	6.7
January 10	1330	3.18	30	36	2.9
January 11	0700	4.98	841	3,090	7,020
January 11	0925	4.33	406	1,240	1,360
January 11	1155	3.85	200	637	344
January 11	1634	3.39	73	168	33
January 12	0705	3.71	152	558	229
January 12	1215	3.51	100	111	30
January 13	0800	3.10	32	27	2.3
January 14	0905	3.18	43	45	5.2
January 14	1235	3.80	177	152	73
January 14	1610	3.56	109	81	24
January 15	1630	3.02	23	17	1.1
January 17	0915	2.90	15	22	.89
January 20	1220	2.86	12	15	.49

TABLE 5 Suspended-Sediment Data for Selected Gaging Stations in Southern California During January-February 1980

Date	Time (hours)	Gage Height (ft)	Discharge (cu ft/s)	Suspended Sediment Concentration (mg/liter)	Suspended Sediment Tons Per Day
January 29	0700	3.90	211	524	299
January 29	0915	3.66	135	274	100
January 29	1650	3.35	66	57	10
January 30	0700	3.19	42	36	4.1
January 30	1615	3.18	39	22	2.3
February 14	1630	2.97	19	11	0.56
February 15	0700	3.60	119	722	232
February 15	0935	3.89	207	392	219
February 15	1645	3.34	64	109	19
February 16	0705	3.36	67	118	21
February 16	1220	7.80	6,700	16,700	302,000
February 17	0735	5.97	2,800	1,980	15,000
February 17	1210	5.45	1,980	1,090	5,830
February 17	1600	5.15	1,600	806	3,480
February 18	0815	6.70	4,660	2,290	28,800
February 18	1220	6.15	3,470	1,670	15,600
February 18	1610	5.87	2,960	1,360	10,900
February 19	0710	6.60	4,890	1,930	25,500
February 19	1210	7.72	7,800	4,180	88,000
February 19	1315	7.50	7,170	7,160	139,000
February 19	1640	7.00	5,860	3,570	56,500
February 20	1620	5.97	3,510	1,590	15,100
February 21	0705	6.18	3,910	1,330	14,000
February 21	1725	5.69	3,000	1,030	8,340
February 22	1650	5.02	1,970	533	2,840
February 23	1630	4.65	1,450	224	877
February 24	1530	4.36	1,130	171	522
February 25	0730	4.26	1,020	113	311

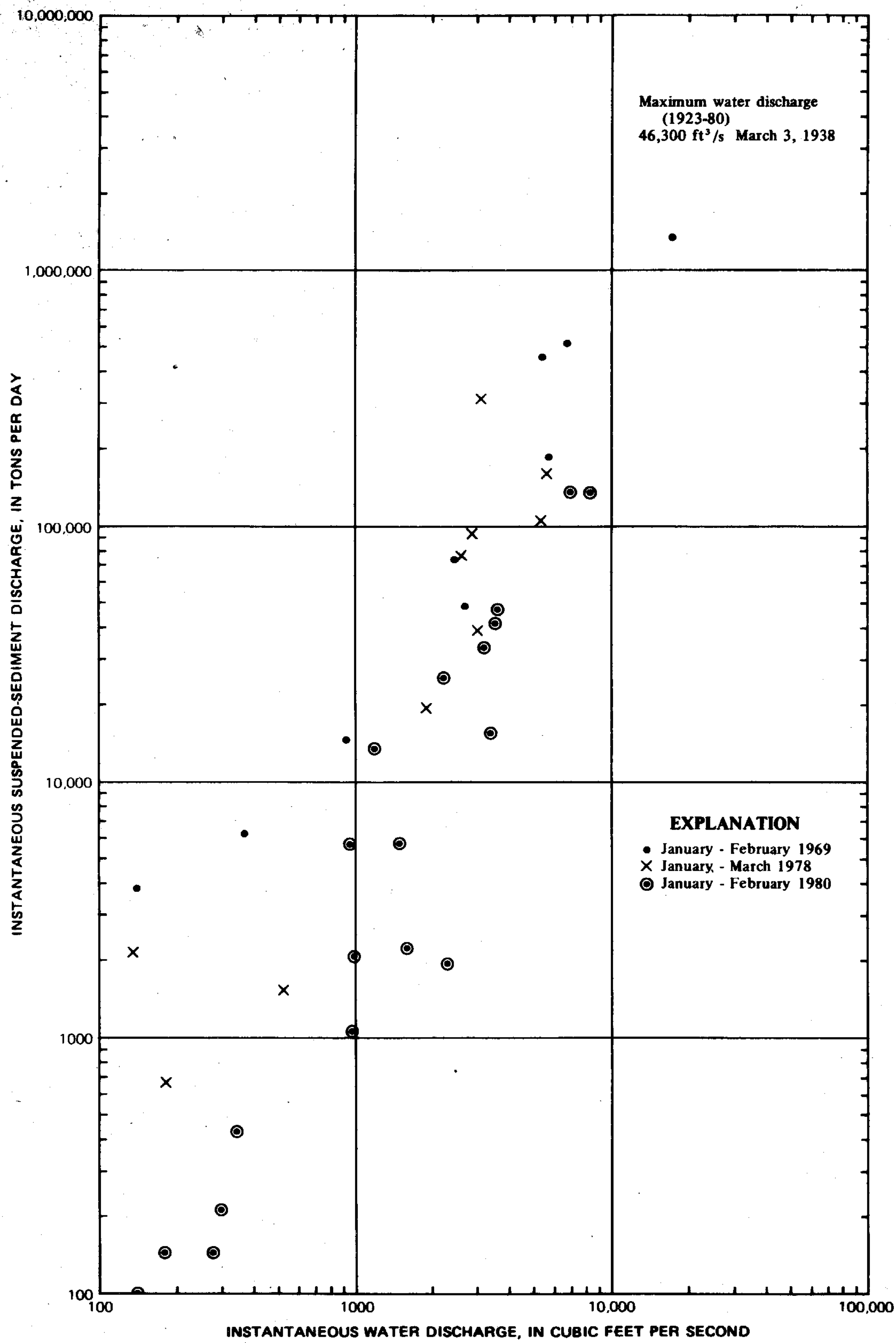


FIGURE 14 Suspended-sediment discharge versus water discharge for selected years at Santa Ana River at Santa Ana.

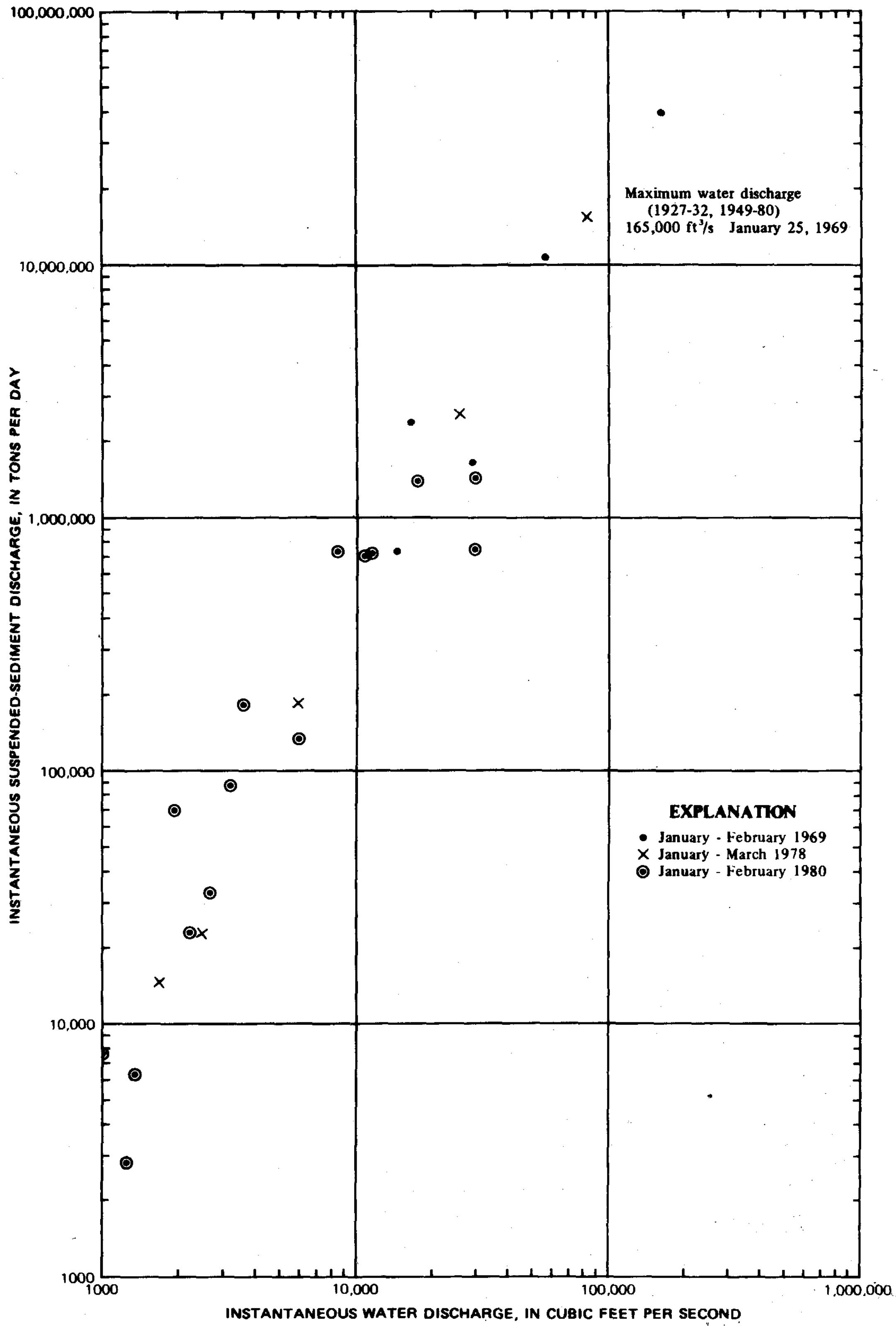


FIGURE 15 Suspended-sediment discharge versus water discharge for selected years at Santa Clara River at Montalvo.

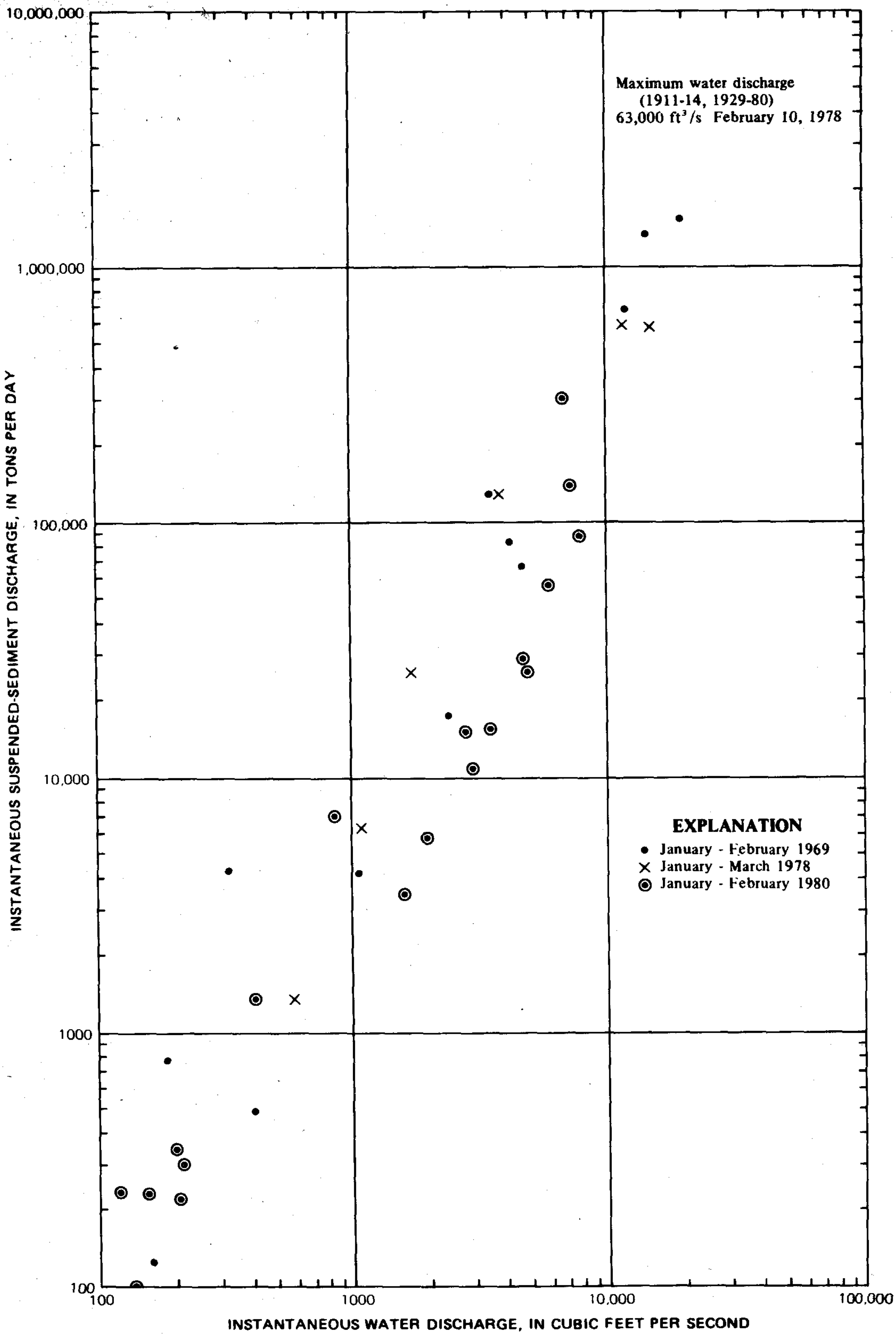


FIGURE 16 Suspended-sediment discharge versus water discharge for selected years at Ventura River near Ventura.

OPERATION AND PERFORMANCE OF CORPS OF ENGINEERS FLOOD CONTROL PROJECTS IN SOUTHERN CALIFORNIA AND ARIZONA DURING 1978-80

by Joseph B. Evelyn

The storms and resultant floods of 1978-80 in southern California and Arizona put many flood control facilities to a reasonably severe test. This paper discusses the operation and performance of Corps of Engineers flood control projects during this period.

The extent that the flood control capability of Corps reservoirs was used in controlling the 1978-80 flood events is examined in terms of the percentage of flood control storage used and the percentage of maximum scheduled reservoir release actually made. Filling frequency relationships at these projects are used to estimate the frequency of occurrence of the 1978-80 flood events.

The effectiveness of reservoir operation is discussed in terms of the reduction in peak discharge inflow compared with maximum reservoir release. The necessity of considering real world circumstances in the planning, design, and operation of flood control works is illustrated by briefly tracing the actual operation of several reservoirs.

A comparison between the first cost plus accumulated operation and maintenance cost for Corps projects and the benefits generated from flood damage reduction during 1978-80 is made. Finally, a summary of pertinent observations is presented, which should prove useful to water resource managers.

INTRODUCTION

The Southwest has recently experienced a series of wet years (1978-80) during which winter precipitation has been well above normal. The resulting floods and mudslides caused extensive damage to homes and businesses, which was vividly portrayed in the news media. Yet the story of how the operation of numerous flood control works effectively prevented the occurrence of catastrophic flooding over large areas of the Southwest was left largely untold. This paper seeks to fill that gap by presenting some quantitative

---

Joseph B. Evelyn is Chief of the Hydrologic Engineering Section with the Los Angeles District Corps of Engineers in Los Angeles, California.

information on the magnitude and frequency of the 1978-80 floods, the reduction in flood magnitude achieved by Corps reservoir projects, and flood damages prevented. Finally, some conclusions and recommendations are put forth that should be of value to both the planners and operators of flood control projects.

LOS ANGELES DISTRICT PROJECTS

Flood control projects operated by the Los Angeles District of the Corps of Engineers lie in three major watersheds: the Los Angeles-San Gabriel River (Figure 1), the Santa Ana River (Figure 2), and the lower Colorado River (Figure 3). Of the 21 reservoirs constructed by the Los Angeles District, this paper will focus on the 11 projects that are gated structures and located above major flood damage centers. Table 1 presents a summary of pertinent characteristics of the projects.

Most of these structures were designed on the basis of controlling a "standard project flood" with a nondamaging reservoir release rate without spillway flow. A standard project flood is a hypothetical flood that would result from the most severe combination of meteorologic and hydrologic conditions considered reasonably characteristic of the region.

FLOOD MAGNITUDES

The storms and resultant floods of 1978-80 in southern California and Arizona put many flood control facilities to a reasonably severe test. Table 2 summarizes the maximum storage attained and the corresponding maximum release associated with Corps projects during 1978-80, as percentages of total available storage and maximum scheduled release. As can be seen in Table 2, the portion of total flood control capability used varied widely from reservoir to reservoir during the same year. For example, during 1980 the percentage of available storage used varied from 9 percent at Carbon Canyon Dam to 78 percent at San Antonio Dam. Table 2 also presents the estimated return period of the reservoir stage, which is referred to as "filling frequency" in the table. Note that the 1980 flood season produced the two most infrequent events in terms of reservoir stage versus frequency. The Sepulveda and Alamo reservoirs reached stages corresponding to recurrence intervals of 60 years and 90 years, respectively.

The portion of total flood control capability used in any particular event is a function of the magnitude of the inflow, constraints on the downstream channel, the basic mode of operation, and adjustments compatible with other water control facilities in the same system. However, the filling frequency curve is a useful indicator of the overall magnitude and recurrence interval of observed events. Figure 4 shows the filling frequency curve at Prado Dam derived from the annual series of flood events since the construction of the dam in 1941. Note that the flood of February 1980 plots at about the 25-year return period. The filling frequency curve tends to lump all the variables pertinent to both the flood and the reservoir operation into a single value.

The Prado Dam volume-frequency curves shown in Figure 5 illustrate in a more definitive manner the magnitude and frequency of the flood inflows. It

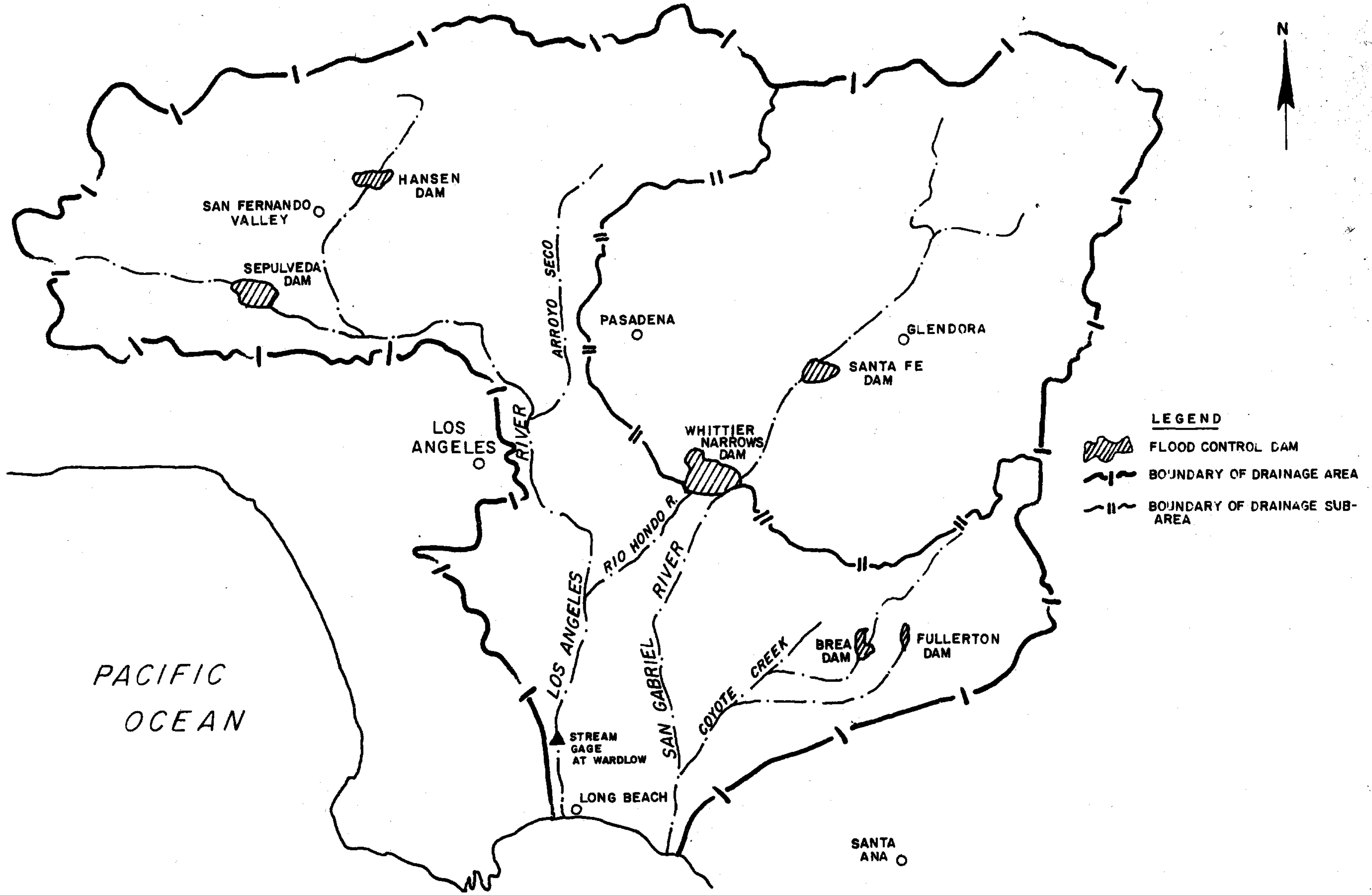


FIGURE 1 Location map of Los Angeles River-San Gabriel River.

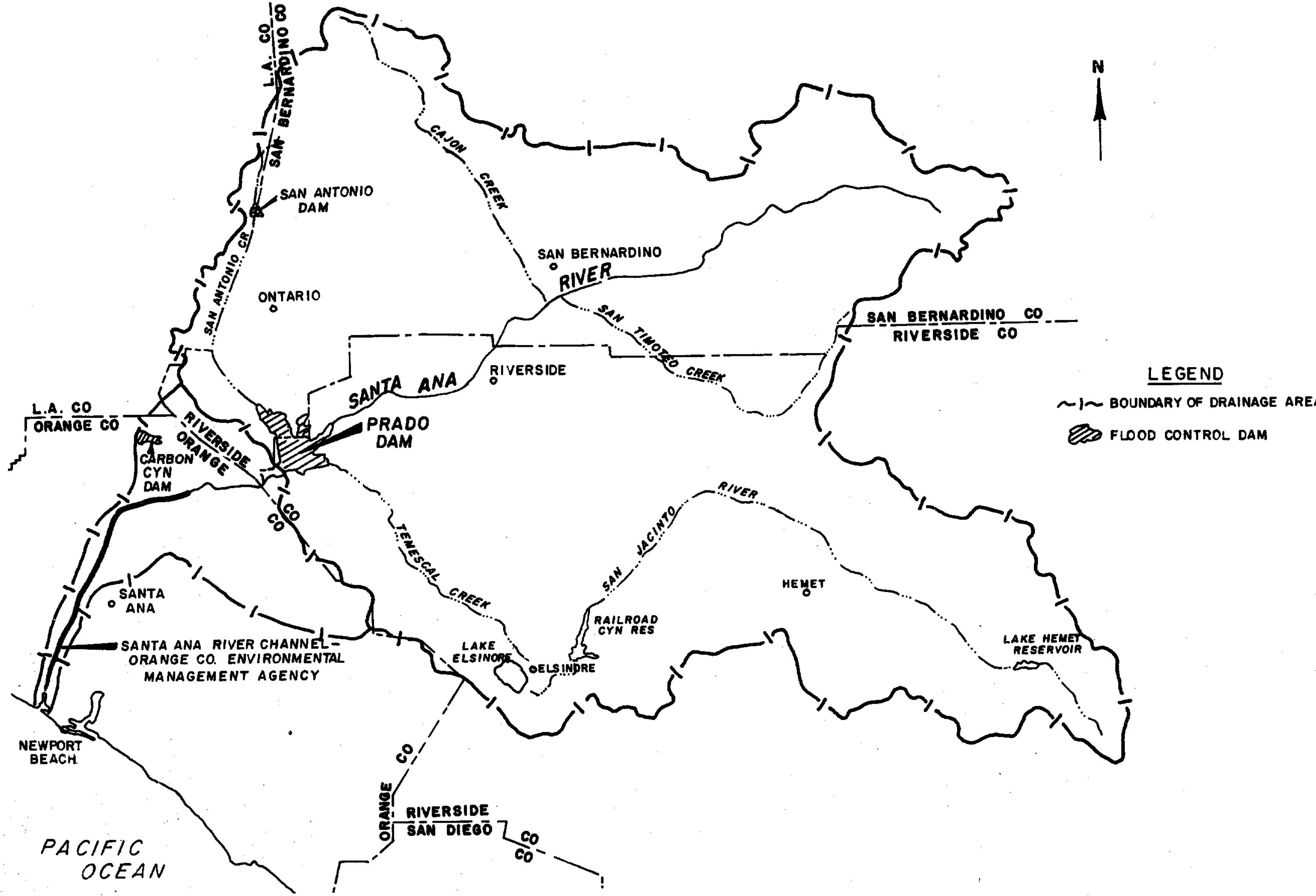


FIGURE 2 Location map of the Santa Ana River.

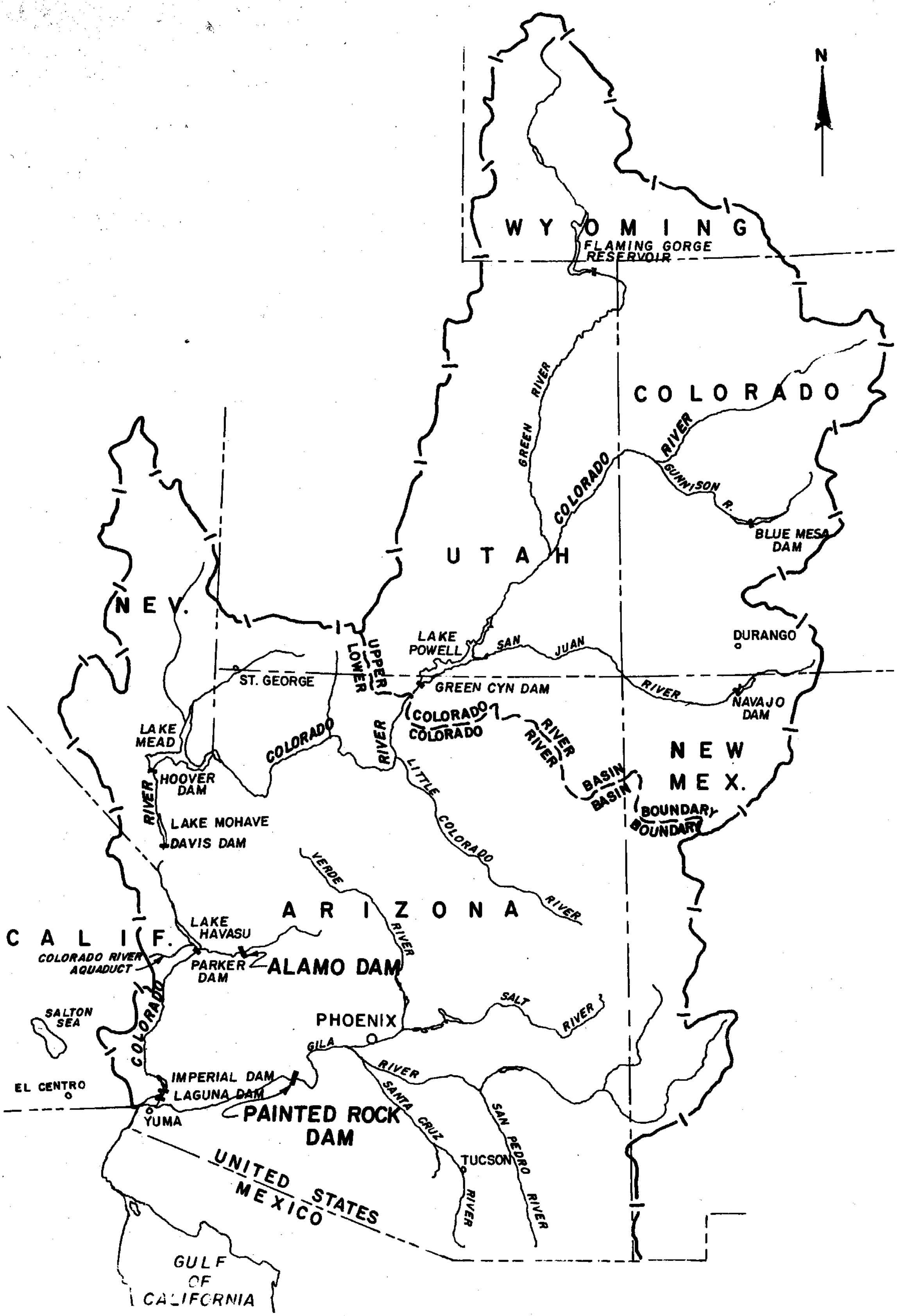


FIGURE 3 Location map of the Colorado River.

TABLE 1 Pertinent Data for Gated Flood Control Reservoirs, Los Angeles District Corps of Engineers

Dam	Stream	Drainage Area (square miles)	Completion Date	Maximum Scheduled Release (cu ft/s)	Storage to Spillway Crest (acre-ft)	Maximum Storage of Record: Storage (acre-ft)	Percent Full	Date
				Los Angeles and San Gabriel River Watersheds				
Sepulveda	Los Angeles River	152	1941	16,500	17,300[a]	11,500	66	Feb. 1980
Hansen	Tujunga Wash	147	1940	23,000	26,090	19,900	56[b]	Jan. 1943
Santa Fe	San Gabriel River	236	1949	41,000	32,600	15,350	44[b]	Jan. 1969
Whittier Narrows	Rio Hondo	554	1957	40,000[c]	36,200[a]	10,240	28[b]	Jan. 1969
Brea	Brea Creek	22	1942	1,400	4,000	870	22	Feb. 1969
Fullerton	Fullerton Creek	5	1941	260	760	520	68	Jan. 1979
				Santa Ana River Watershed				
San Antonio	San Antonio Creek	27	1956	8,200	7,650	5,950	78	Feb. 1980
Prado	Santa Ana River	2,233	1941	5,165	198,000	111,000	56	Feb. 1980
Carbon Canyon	Carbon Creek	19	1961	1,050	6,600	720	11	Mar. 1979
				Lower Colorado River Watershed				
Alamo	Bill Williams River	4,770	1968	7,000	1,045,000	724,000	69	Feb. 1980
Painted Rock	Gila River	50,800	1959	22,500	2,492,000	1,849,000	74	Mar. 1980

[a]Storage at the top of flood pool (is above spillway crest due to gated spillway).

[b]Percent full based on storage capacity on date of flood.

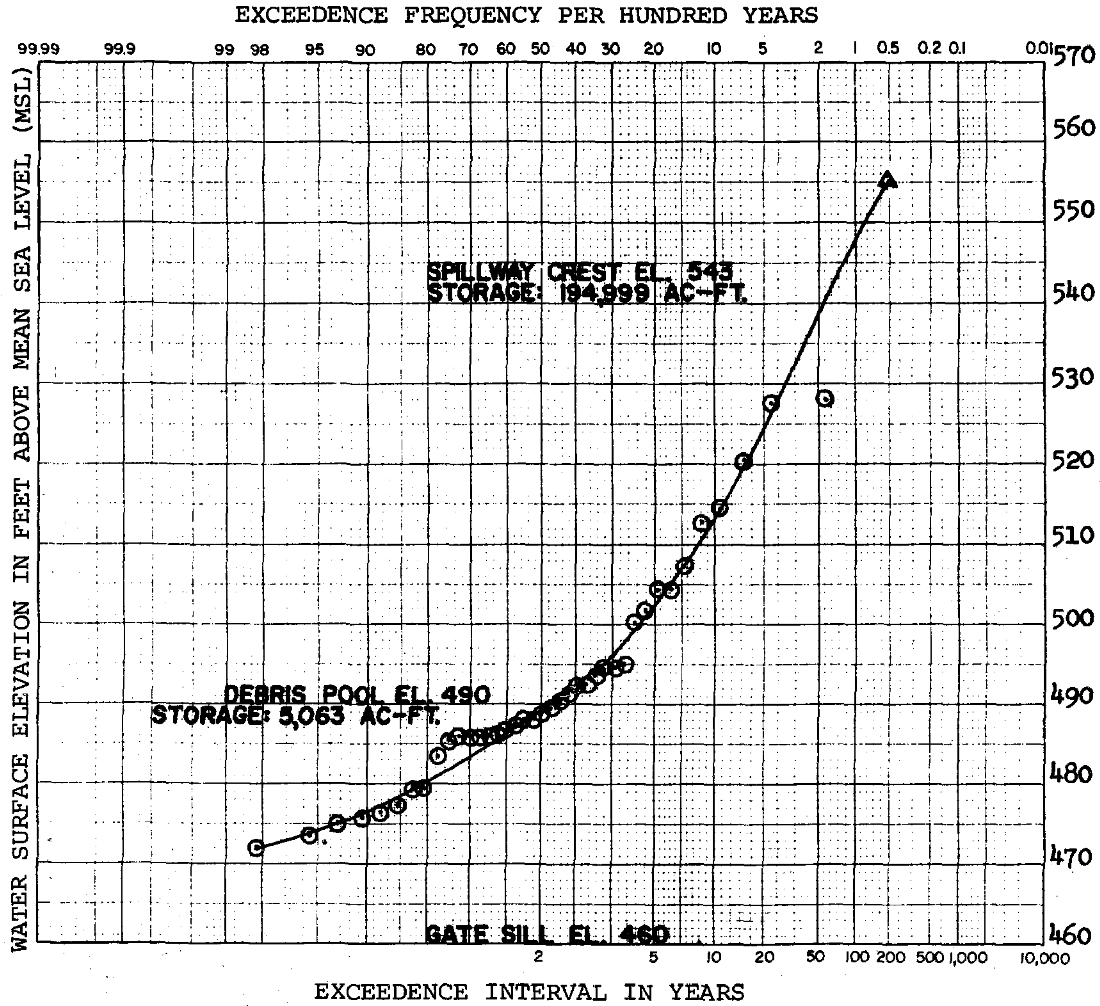
[c]Rio Hondo outlet only. An additional 5,000 cu ft/s is scheduled for the San Gabriel River outlet.

TABLE 2 Magnitude of 1978-80 Flood Events at Gated Flood Control Reservoirs, Los Angeles District Corps of Engineers

Dam	Storage to Spillway Crest (acre-ft)	Maximum Scheduled Release (cu ft/s)	Water Year 1978			Water Year 1979			Water Year 1980		
			Percent Full	Filling Frequency (years)	Percent Maximum Scheduled Release	Percent Full	Filling Frequency (years)	Percent Maximum Scheduled Release	Percent Full	Filling Frequency (years)	Percent Maximum Scheduled Release
				Los Angeles and San Gabriel River Watersheds							
Sepulveda	17,300[a]	16,500	30	14	76	6	3	59	66	60	92
Hansen	26,090	23,000	31	32	57	11	5	3	22	17	16
Santa Fe	32,600	41,000	17	4	35	2	2	1	25	6	45
Whittier Narrows	36,200[a]	40,000	14	17	80	11	2	54	19	40	89
Brea	4,000	1,400	4	5	72	5	6	84	18	25	76
Fullerton	760	260	42	10	108	68	40	110	36	9	110
				Santa Ana River Watershed							
San Antonio	7,650	8,200	37	40	26	9	5	1	78	n/a	25
Prado	198,000	5,165	40	18	44	15	7	10	56	25	120
Carbon Canyon	6,600	1,050	11	13	31	11	13	21	9	9	49
				Lower Colorado River Watershed							
Alamo	1,045,000	7,000	32	10	4	49	25	8	69	90	57
Painted Rock	2,492,000	22,500	15	n/a	1	65	n/a	13	74	n/a	20

Note: n/a = not available.

[a]Storage at top of flood pool (is above spillway crest due to gated spillway).



LEGEND

⊙ Median plotting positions for period 1942-80 (N = 39) recorded water surface elevations greater than 472 ft MSL

△ Standard project flood, elevation 555 ft, based on gross storage capacity

Drainage area = 2,255 sq miles

FIGURE 4 The filling frequency curve at Prado Dam, partial duration.

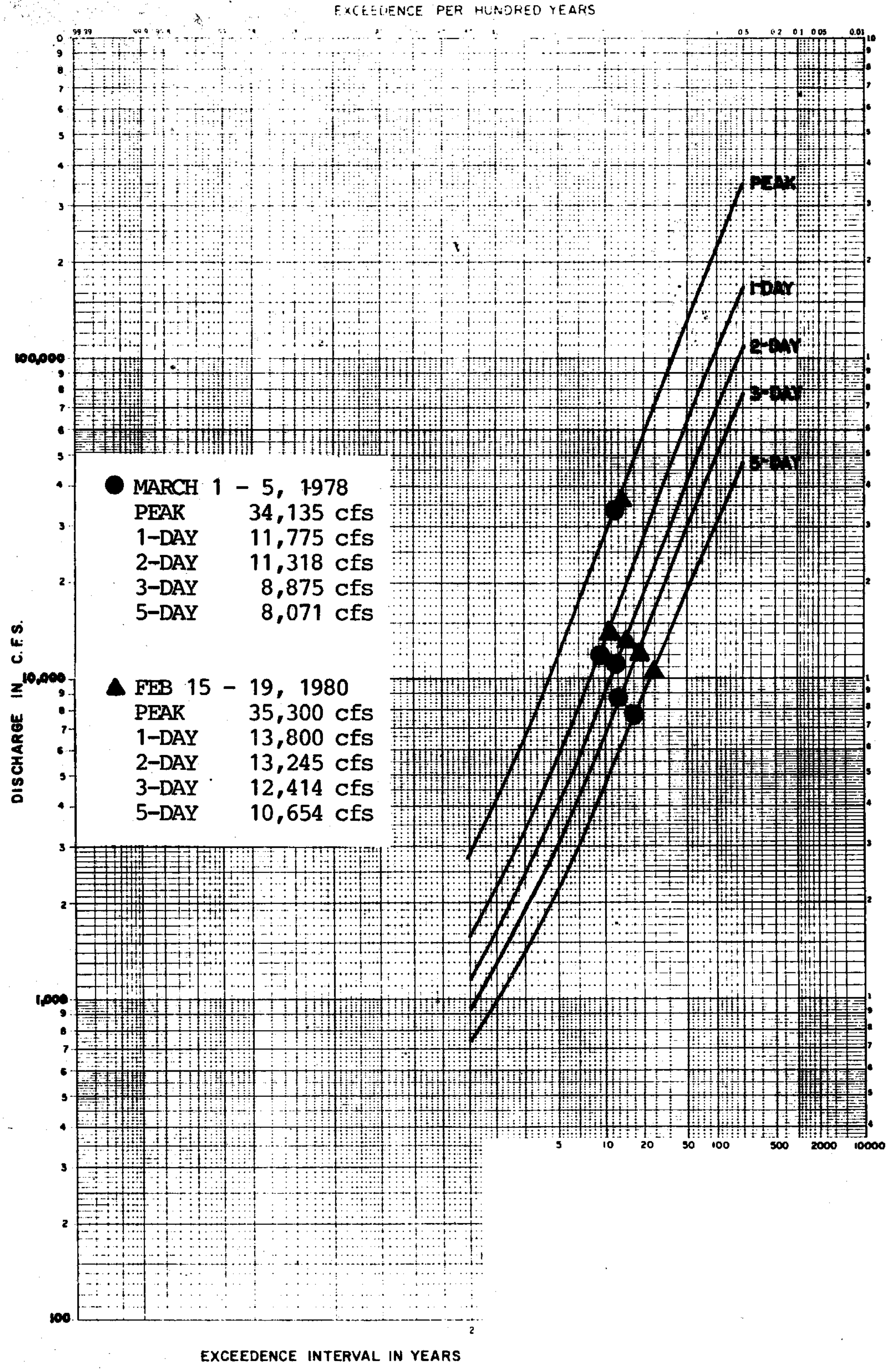


FIGURE 5 Volume-frequency curves for the Santa Ana River at Prado Dam (drainage area = 2,255 sq miles).

should be noted that the recurrence interval of a single flood event varies with the duration selected. In Figure 5 it can be seen that the February 1980 flood inflow at Prado Dam was a 14-year event with respect to instantaneous peak discharge, an 11-year event for a one-day duration, and a 30-year event for a five-day duration. The filling frequency relationship (Figure 4) smoothes this range of return periods versus duration to indicate that a 25-year return period is representative of the entire February 1980 flood event.

RESERVOIR OPERATION

A measure of the effectiveness of a reservoir facility in controlling flood runoff is the degree of reduction of peak reservoir inflow as compared with maximum reservoir release. Table 3 summarizes the reductions in peak inflows achieved by gated Los Angeles District reservoirs during 1978-80. Note that substantial reductions were achieved at most reservoirs, particularly those having natural unimproved downstream channels, such as Prado, Alamo, and Painted Rock dams. Where high-capacity, improved channels were available, such as at Whittier Narrows Dam, reductions in peak reservoir inflows were typically much smaller, since large releases could be made safely from these structures.

The remainder of this section describes the operation of several of the Corps reservoirs discussed above, in order to convey in more detail an understanding of the actual operations and the decision making involved. The importance of adequate downstream channel capacity, forecasting of projected reservoir inflows, and timely exchange of information with other agencies operating reservoirs in the same system can be inferred in the following discussions.

Prado Dam operation during January-August 1980 is shown graphically in Figure 6. During the large flood inflow in mid-February, reservoir releases were held in the range of 5,000 to 6,000 cu ft/s, except for a brief cutback to evaluate the condition of the downstream channel and make emergency channel repairs. Similar temporary cutbacks were made in late March and April, until in mid-May it was possible to reduce releases to 300 cu ft/s, which could be recharged entirely to groundwater by the Orange County Water District, thereby maintaining a dry lower river channel to facilitate the repair of damaged portions of the channel through Orange County. The reservoir had to be dry in mid-August to permit seismic investigation of the Prado Dam embankment as well as to perform needed gate maintenance.

Sepulveda Dam operation during the February 16, 1980, flood event is depicted in Figure 7. Note that reductions in release were required between 1600 and 2100 hours so as not to exceed the downstream channel capacity of 16,500 cu ft/s. These adjustments were made on the basis of reports by channel observers in the field at that time.

Beginning in 1978 significant inflows occurred at Painted Rock Dam each winter following large spills from upstream water conservation reservoirs. Figure 8 illustrates the operation of Painted Rock Reservoir during 1978-80.

TABLE 3 Reduction of Peak Inflows at Gated Flood Control Reservoirs, Los Angeles District Corps of Engineers

Dam	Drainage Area (square miles)	Water Year 1978			Water Year 1979			Water Year 1980		
		Peak Inflow (cu ft/s)	Maximum Release (cu ft/s)	Percent Reduction of Peak Inflows	Peak Inflow (cu ft/s)	Maximum Release (cu ft/s)	Percent Reduction of Peak Inflows	Peak Inflow (cu ft/s)	Maximum Release (cu ft/s)	Percent Reduction of Peak Inflows
Los Angeles and San Gabriel River Watersheds										
Sepulveda	152	25,670	13,190	49	16,410	9,680	41	62,000	15,100	76
Hansen	147	49,800	13,120	74	1,785	740	59	9,300	4,560	51
Santa Fe	236	17,610	14,160	20	420	180	57	15,000	19,200	0
Whittier Narrows	554	37,260	31,850	15	16,700	21,600	0	41,550[a]	35,600[a]	14[a]
Brea	22	1,540	1,010	34	1,580	1,170	26	2,250	1,060	53
Fullerton	5	1,300	285	78	1,328	285	79	1,050	285	73
Santa Ana River Watershed										
San Antonio	27	2,070	1,980	4	275	80	71	2,300	2,060	11
Prado	2,233	34,100	2,250	93	5,885	510	91	42,200	6,000	86
Carbon Canyon	19	1,030	325	68	637	225	65	1,080	520	52
Lower Colorado River Watershed										
Alamo	4,770	78,000	1,500	98	67,000	1,500	98	85,000	4,000	95
Painted Rock	50,800	128,800	250	99.8	128,100	3,000	98	200,000[b]	4,500	98

[a]Rio Hondo outlet only.

[b]Approximate.

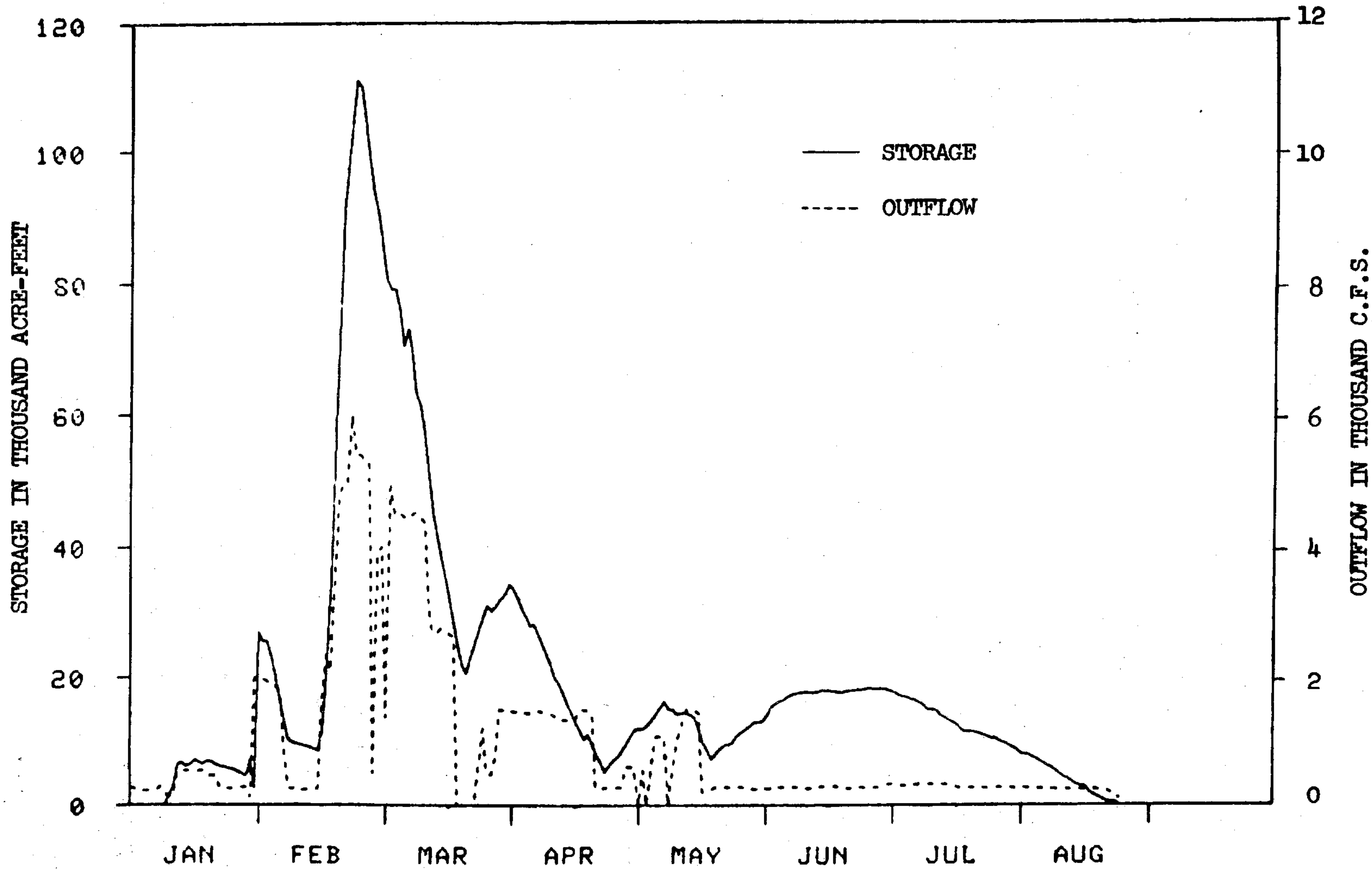


FIGURE 6 Storage and outflow at Prado Reservoir during the 1980 floods.

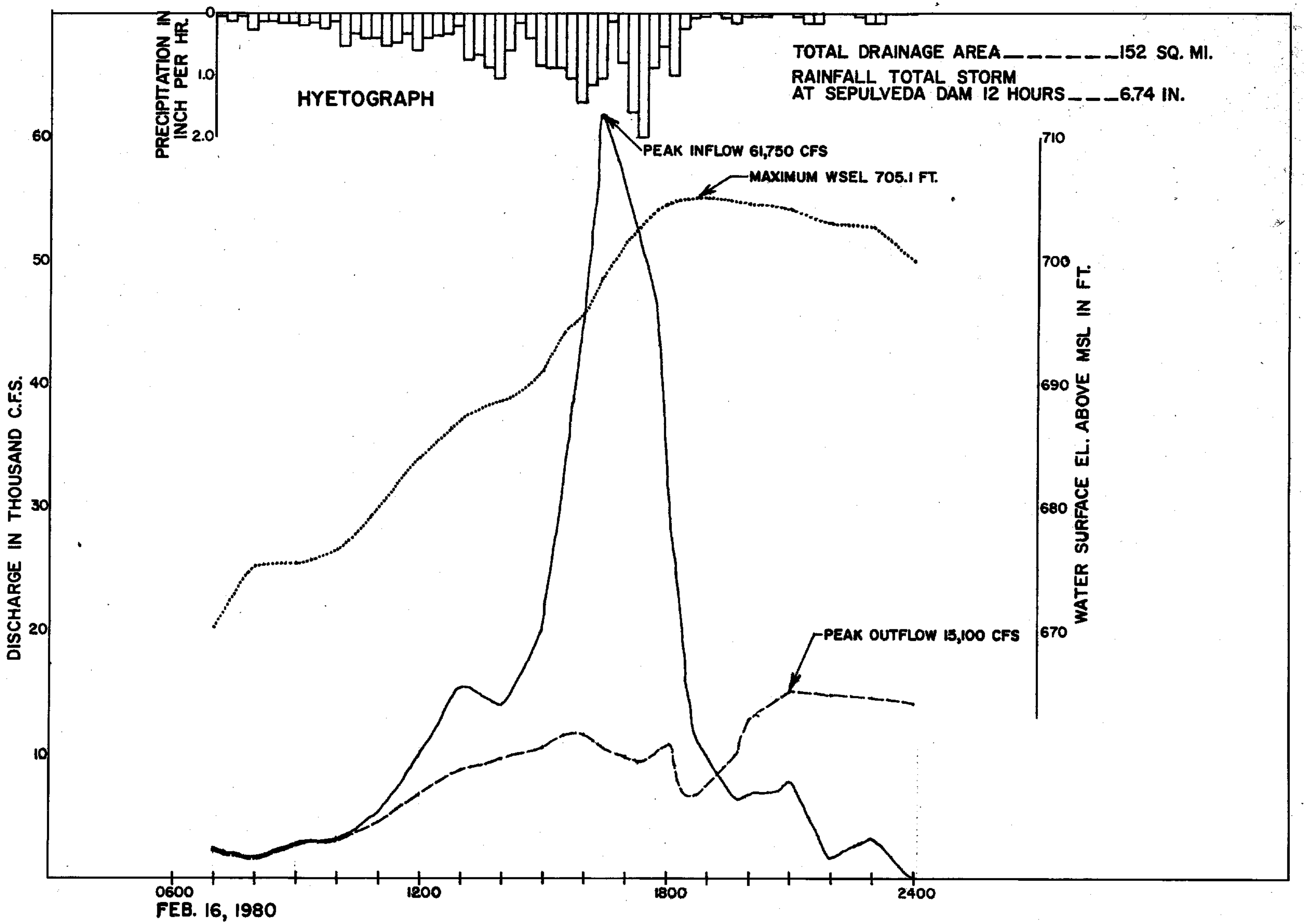


FIGURE 7 Operation of Sepulveda Dam on February 16, 1980.

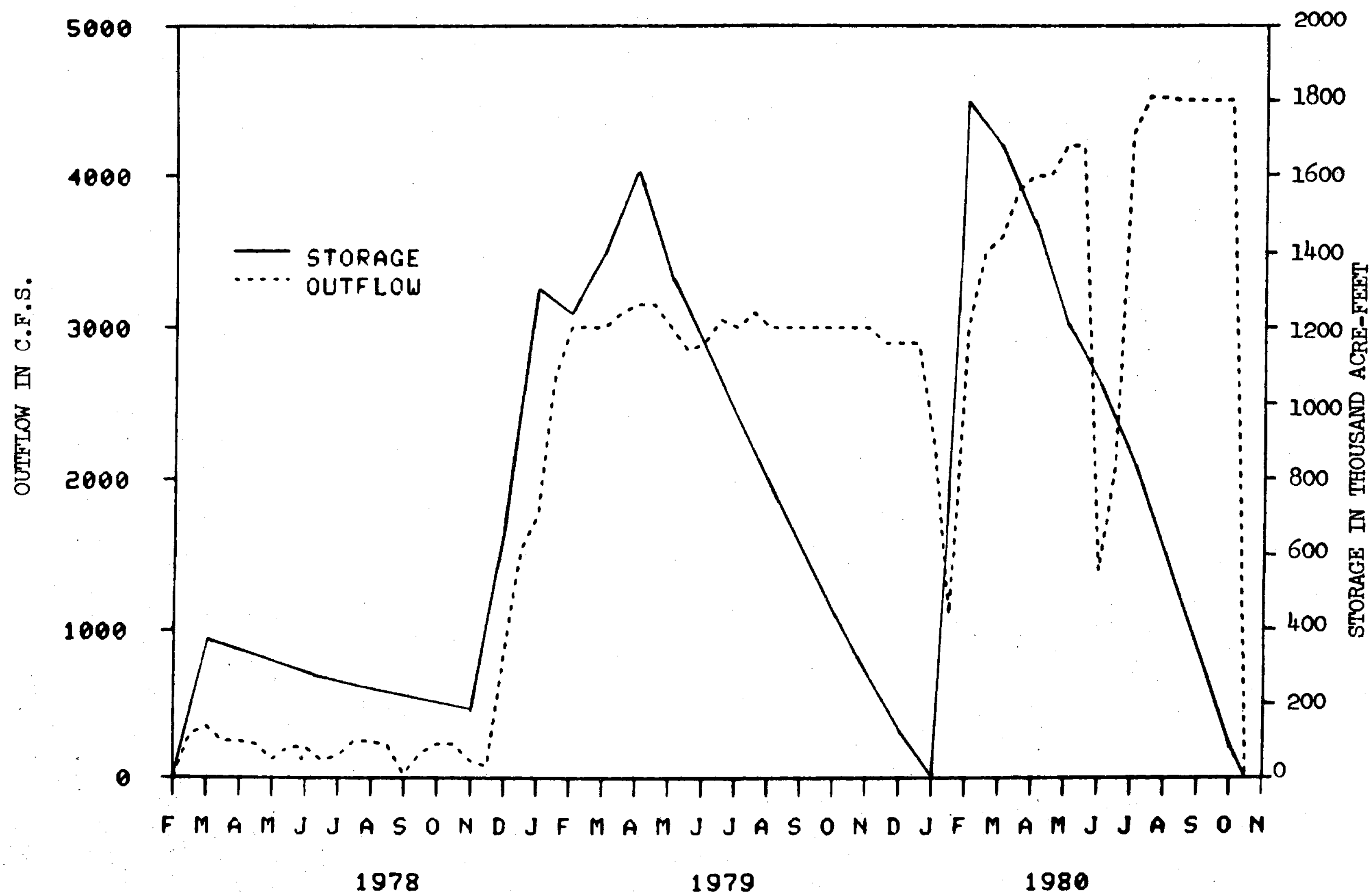


FIGURE 8 Storage and outflow curves for Painted Rock Dam, March 1978-October 1980.

Due to many years of little or no flow on the Gila River downstream of Painted Rock Dam, and to use of the floodplain by agricultural interests, adequate downstream channel capacity is no longer available to make scheduled flood control releases. Although the maximum scheduled flood control release is 22,500 cu ft/s, releases were constrained to a maximum of 250 cu ft/s in 1978, about 3,000 cu ft/s in 1979, and about 4,800 cu ft/s in 1980. Forecasts of probable inflow were essential to accommodating such a constrained reservoir operation.

LOWER LOS ANGELES RIVER

Thus far, attention has been focused on reservoir projects. The functioning of the channels that are part of the Los Angeles-San Gabriel River system is also pertinent to the discussion of project operation and performance. Annual peak discharge frequency curves for the Los Angeles River near the Wardlow stream gage (see Figure 1 for location) are presented in Figure 9. The leveed Los Angeles River channel at this location was designed to carry 146,000 cu ft/s. The February 16, 1980, flood of 125,000 cu ft/s, the largest flood of record, required using 86 percent of the design channel capacity.

Generally, improved channels (concrete or rock lined) in the Los Angeles-San Gabriel system performed well, with only minor problems and little or no damage. The most significant problem observed was localized water surface disturbances caused by large storm drain side inflows. Standing waves, in some locations several feet higher than the water surface upstream and downstream of the disturbance, required throttling back reservoir releases in order to prevent possible damage to the channel itself. The Los Angeles River about 1 mile downstream of Sepulveda Dam (Figure 10) vividly illustrates this situation.

Two discharge frequency relationships for the Los Angeles River near the Wardlow gage are presented in Figure 9. A graphical curve using median plotting positions is drawn through the annual peak discharges for the record beginning in 1941. However, since the drainage area has been urbanizing, the change in percentage of impervious cover and drainage system improvements (e.g., storm drains) have greatly increased the efficiency of runoff. This is illustrated in Figure 11, which depicts the change in mean annual peak discharge versus time from 1941 to the present. The points plotted in Figure 11 represent the mean of the annual peak discharges occurring in the 10-year period immediately prior to the year at which the point is plotted. The mean peak discharge for the 10-year period 1941-50 was determined to be about 17,000 cu ft/s. The mean peak discharge for the period 1970-80 had increased to about 56,000 cu ft/s. In order to reflect this change in watershed response due to urbanization, the mean peak discharge for 1970-80 was used in conjunction with the upper end of the best-fit curve in Figure 9 to draw a graphical estimate of the present discharge frequency relationship, shown as a dashed line in Figure 9. Note that the 125,000-cu ft/s peak discharge plots at about the 50-year return period.

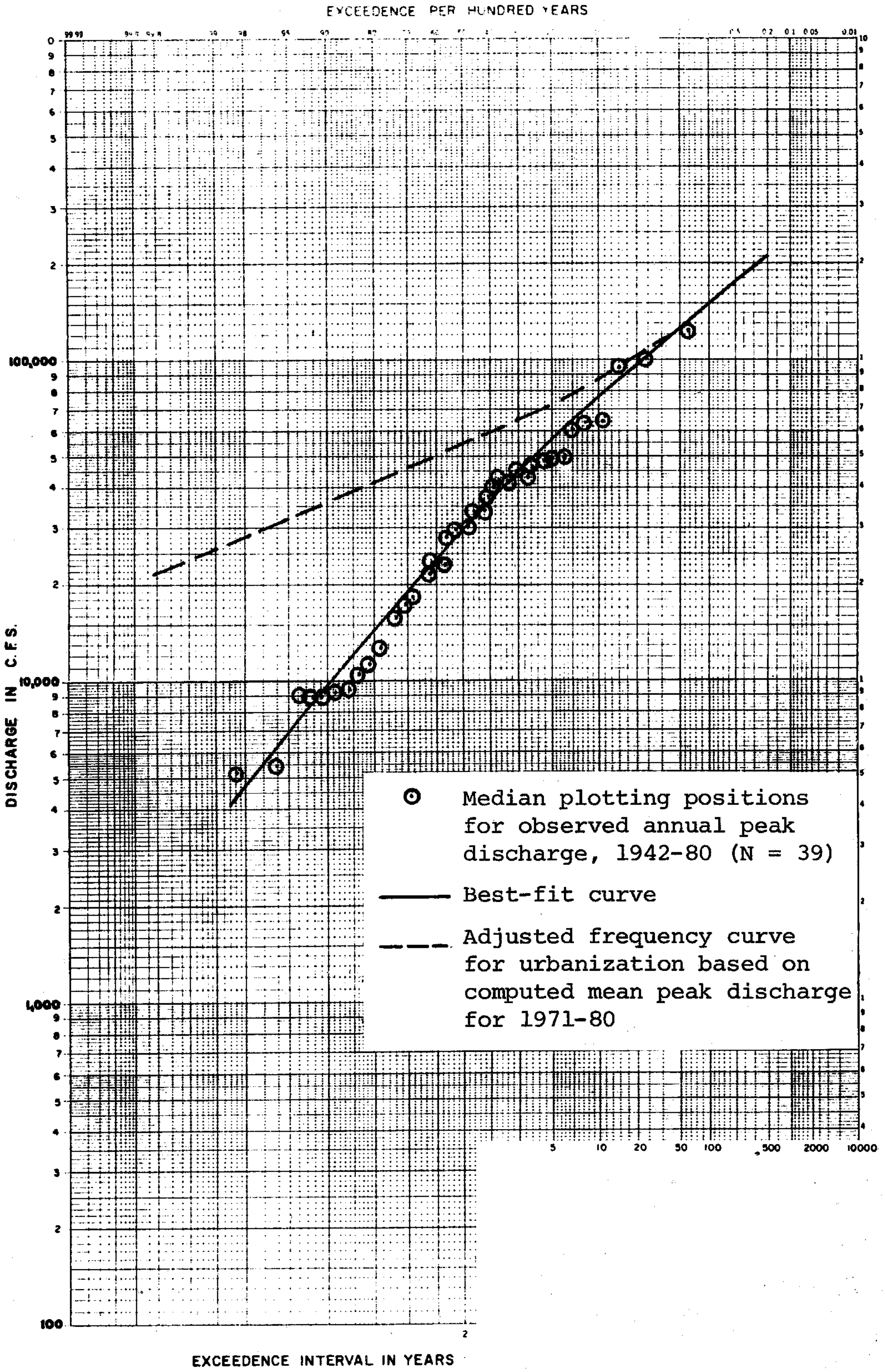


FIGURE 9 Discharge frequency curve for the Los Angeles River at Wardlow Road.

FIGURE 10 Looking downstream on the Los Angeles River near Kester Avenue (approximately 1 mile downstream of Sepulveda Dam) on February 16, 1980. Note the standing wave resulting from side weir overflow. (Photo courtesy of Los Angeles County Flood Control District.)

COSTS AND BENEFITS

The discussion of the performance of the projects would not be complete without addressing the benefits resulting from reduction in potential flood damage achieved through the operation of the projects versus their cost to the public. Table 4 summarizes the original cost of the projects plus the accumulated annual operation and maintenance costs, along with flood damages prevented, using present price levels. Dollar damage estimates were developed by the Corps' Economics Section based on available damage versus discharge relations for the condition of no projects in place. The comparison of flood

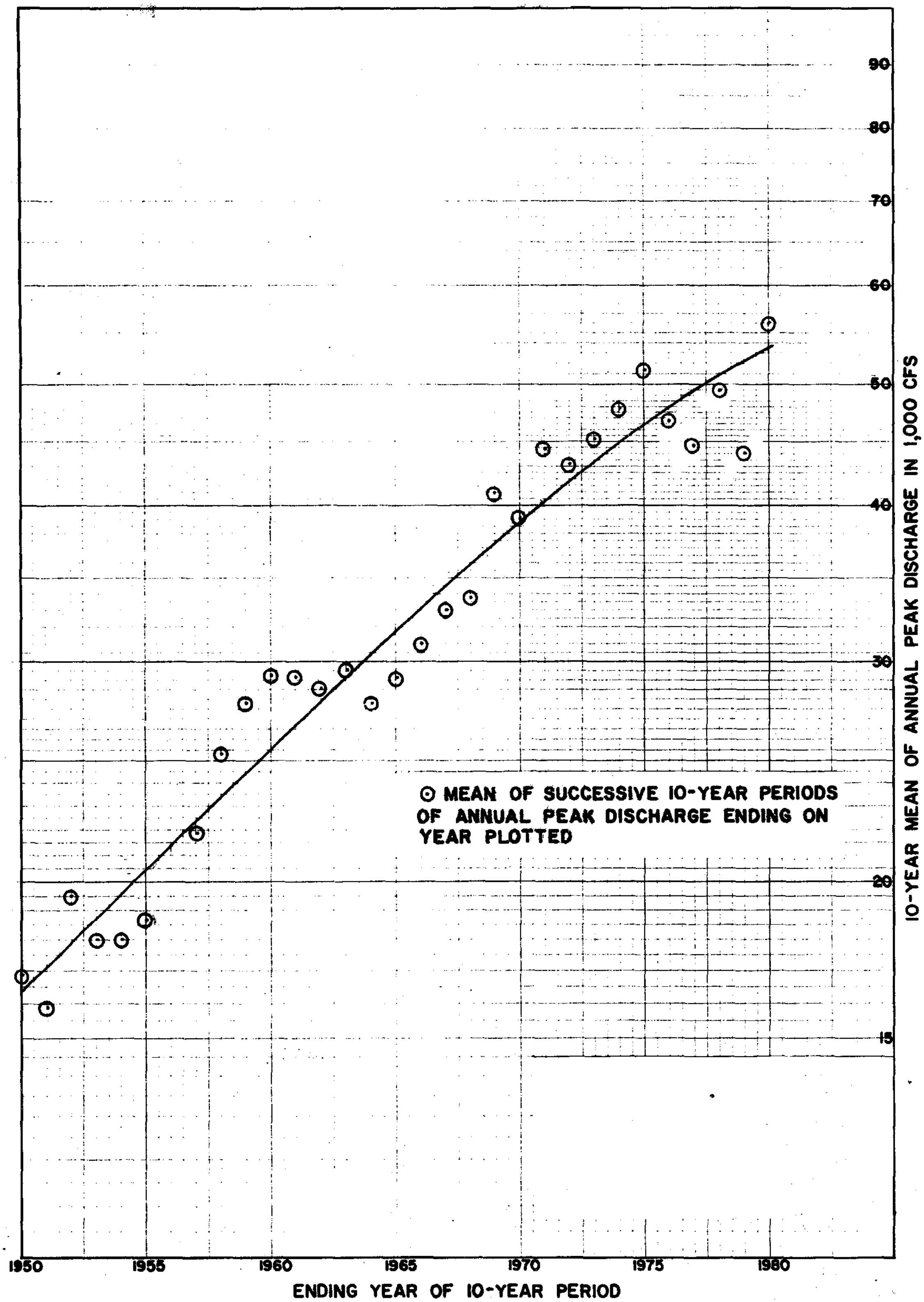


FIGURE 11 Mean of annual peak discharge versus time for the Los Angeles River near Wardlow.

TABLE 4 Costs and Benefits for Gated Flood Control Reservoirs and Selected Channels (thousands of dollars), Los Angeles District Corps of Engineers

	Costs[a]			Damages Prevented in Water Year			
Project	Construction	Accumulated Operation and Maintenance to Date	Total Construction and Operation and Maintenance	1978	1979	1980	1978-80
			Los Angeles and San Gabriel River Watersheds				
Sepulveda	7,900	3,200	11,100	140,000	0	290,000	440,000
Hansen	12,100	2,400	14,500	280,000	0	44,000	320,000
Santa Fe	19,600	2,400	22,000	210,000	0	200,000	400,000
Whittier Narrows	44,700	4,100	48,800	97,000	0	71,000	170,000
Brea	1,200	2,200	3,400	5,100	5,100	7,300	18,000
Fullerton	2,800	1,700	4,500	2,000	2,000	1,500	5,500
Channel System	314,900	13,500	328,400	480,000	0	900,000	1,400,000
			Santa Ana River Watershed				
San Antonio	7,000	2,000	9,000	5,000	0	5,200	10,000
Prado	12,500	2,900	15,400	180,000	0	220,000	400,000
Carbon Canyon	7,700	1,500	9,200	6,200	1,200	6,200	14,000
			Lower Colorado River Watershed				
Alamo	16,900	2,000	18,900	4,000	8,300	11,000	23,000
Painted Rock	20,000	3,000	23,000	82,000	82,000	120,000	280,000
Total	467,300	40,900	508,200	1,491,300	98,600	1,876,200	3,480,500

[a]Not adjusted for inflation.

damages prevented and the cost of the flood control works indicates that most of the projects paid for themselves several times over during the period 1978-80.

SUMMARY

1. The magnitude and frequency of occurrence of the 1978-80 floods varied widely among the Corps projects. However, on the basis of the filling frequency relationships derived for each project, most of the observed flood events had recurrence intervals of less than 50 years, and all were less than 100-year events.

2. Water resource managers should recognize the need to make realistic assumptions with respect to usable channel capacity below flood control reservoirs during both design and operation.

3. During flood control operation there is a real need for rapid and sure exchange of information about the operation of projects within a river system and about the impacts and problems resulting from operating the system.

4. Flood forecasting of inflows is essential to optimizing the operation of flood control facilities.

5. Deviations from the original reservoir operation plan, commonly based solely on control of a particular design flood, are frequently necessary to accommodate real world circumstances.

6. Corps flood control facilities have been cost effective measures in the Southwest.

REFERENCES

Corps of Engineers, Los Angeles District, 1940 through 1980, unpublished reservoir operation records, Los Angeles.

U.S. Geological Survey, Water Years 1942 through 1980, Water Resources Data for California, Los Angeles.

FLOODFLOWS IN MAJOR STREAMS IN VENTURA COUNTY

by Dolores B. Taylor

Several floodflows of special importance occurred in Ventura County during the storms of February 1980. The most devastating was the breach of the Calleguas Creek levee, which released an estimated 24,000 acre-ft of water and 1.7 million tons of sediment onto the Oxnard plain. At the Point Mugu Naval Base, which was in the path of the floodwaters, damages to 470 residences totaled some $9 million.

To warn the residents of Point Mugu of future flooding, the Navy, the National Weather Service River Forecast Center, and the Ventura County Flood Control District have set up a flood warning system for Calleguas Creek. This system combines rain and stream gages, runoff models, and sediment-monitoring procedures to provide advance notice that flood conditions are imminent.

Several other large floodflows in Ventura County were related to the Creek fire of September 1979, which blackened 33,000 acres. The Canada Larga watershed produced a wave of debris-laden water that demolished buildings in its path. In East Ventura, Arundell Barranca overtopped its west bank for the first time in memory and swept through the nearby streets.

Along with the usual collage of problems associated with intense storms on saturated watersheds, two primary events affected Ventura County during the storms in February 1980.

The most devastating in terms of the number of people affected, the property damaged, and the manpower required to mitigate the event was the Calleguas Creek levee break. Some $9 million of damage was done at Point Mugu, with nearly 3,000 people suffering from the psychological stress of evacuation, the loss of household goods, and the fear that this could happen again.

Approximately 1,500 acres of valuable agricultural land were covered by floodwaters ranging from 2 to 5 ft in depth. An estimated 24,000 acre-ft of

---

Dolores B. Taylor is Surface Water Hydrologist with the Ventura County Flood Control and Water Resources District in Ventura, California.

floodwaters escaped through the levee breach from 6:00 p.m., February 16, 1980, to 2:00 p.m., February 20, 1980. After the water receded, approximately 450 acres of farmland were buried by silt from 4 in. to 4 ft deep.

The second event that was a cause for major concern was the storm's impact on the upland areas burned in the 33,000-acre Creek fire that occurred in September 1979. Thirteen large watersheds, draining to the Ventura River and lower Santa Clara River, were 50 to 90 percent burned. The intense heat from the fire turned tree stumps into white powder and left no sign of a prior orchard. During subsequent storms, culverts at the canyon outlets captured debris while silt and ash swept into channels below, creating the highest peaks on record. Debris production formulas developed by Scott and Williams (1974) were used to predict the amount of bulking of runoff from the burned areas. The estimates of tremendous debris production were borne out by subsequent debris entrapment in control structures. Damage to flood control facilities amounted to $2 million, resulting from the 15-year rainfall event (six hours of rainfall) and the consequent peak discharge, which was in excess of an event with a 100-year return period.

CALLEGUAS CREEK LEVEE BREAK

Chronology of Event

On Saturday, February 16, 1980, at approximately 6:00 p.m., a conscientious flood control hydrographer called into the County Storm Center to report that sheet flow was lapping over the east levee above the USGS gage at Calleguas Creek near Camarillo State Hospital. The gage height reported at that time corresponded to a channel discharge of 25,300 cu ft/s--a 50-year event and over twice the original design flow capacity. To prove herself in charge, Ma Nature compounded the problem with a very high tide and onshore wind waves. Ordinarily, the waves near Point Mugu approach obliquely so that the full effect on the outflow is not felt. That night, however, wave action was perpendicular to the shore, driven by the third in a series of storms.

After dark, ranchers in the area reported some breakout near Highway 1 where Revolon Slough and Calleguas Creek merge, but the full comprehension of what had occurred did not come until dawn. Then, 1,500 acres between a major breach in Calleguas' levee and Highway 1 looked like a muddy lake.

Revolon Slough's levees, rising several feet above the surrounding agricultural land, blocked the seaward path of the escaping floodwaters. Because of rising floodwaters at the junction of the two levees above Highway 1, the Revolon Slough east levee gave way, followed by overtopping on its west bank. Calleguas Creek's floodwaters poured through the escape channels, filling in the low-lying area (see Figures 1 and 2). Highway 1, nearly 5 ft above the surrounding ground, has a low point directly opposite base housing at Point Mugu. This sag acted like a 1,200-ft-long weir, with floodwaters sheeting across and into base housing. Streets and yards began to flood without warning. Base operations at several locations came to a standstill and civilian employees were sent home. At the worst point, 30 in. of water was standing inside many homes. Families grabbed what few things they could

FIGURE 1 This aerial photograph shows the levee break on Calleguas Creek and construction crews trying to rechannel the escaping water. A total of 24,000 acre-ft of water flowed through the breach before it was repaired, and 450 acres of agricultural land were buried by silt from 4 in. to 4 ft deep. (Photograph by Western Aerial, February 22, 1980.)

and were quickly evacuated to nearby Port Hueneme to await the recession of floodwaters. Of the 550 housing units at Point Mugu, 470 units received some flood damage.

After the levee breached, channel headcutting occurred upstream of the breach, lowering the channel bottom and contributing additional sediment to be deposited on flooded agricultural areas.

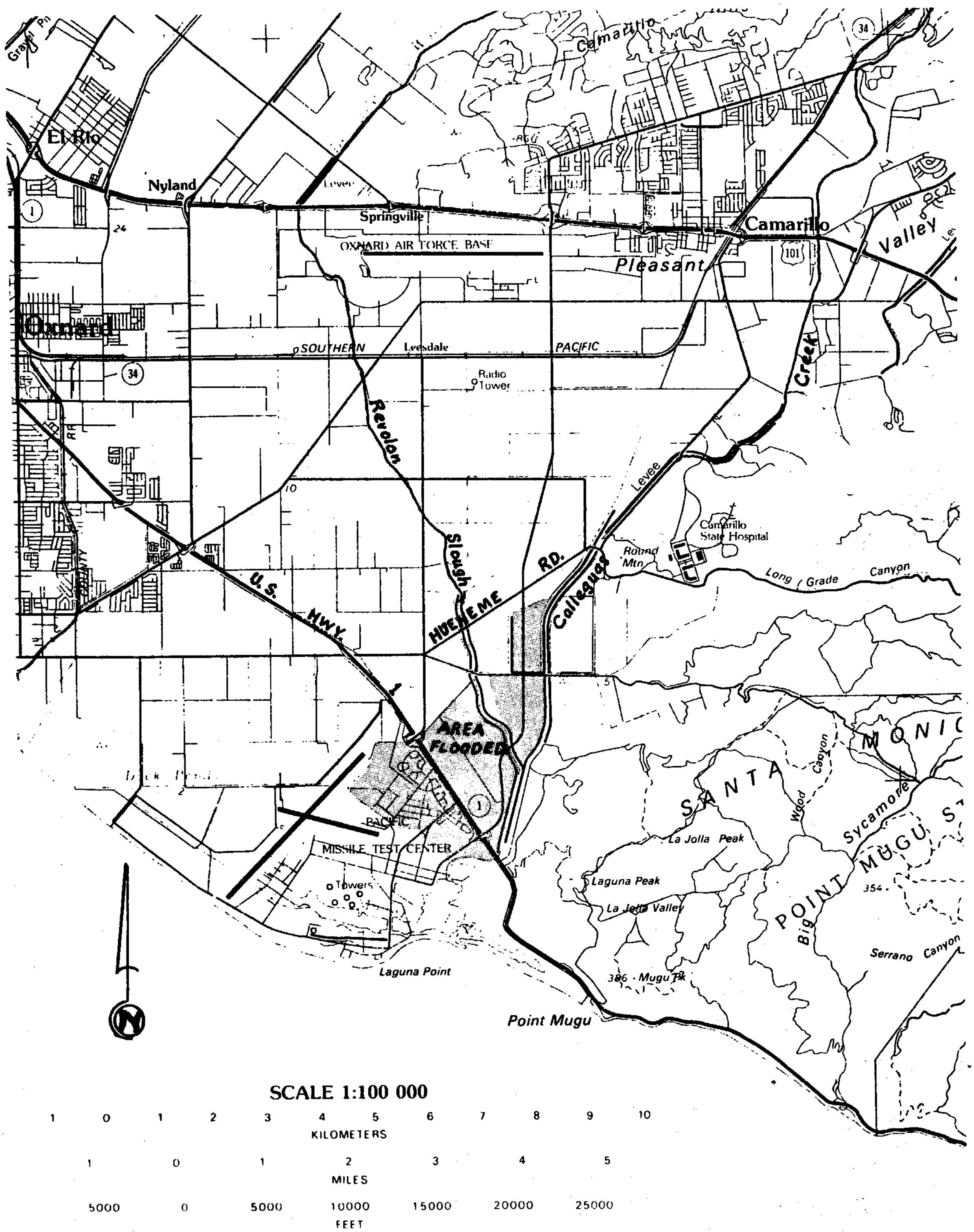


FIGURE 2 Location map showing the area flooded by the Calleguas Creek levee break on February 16, 1980. The area south of U.S. Highway 1 encompasses the Point Mugu base housing.

Below the breach the streambed was plugged with sediment. This sediment had to be removed to restore some channel capacity and prepare for the next storm. An emergency contractor and Navy construction forces (Seabees) from Port Hueneme attacked the breach with bulldozers. Calculations of the required distance to bulldoze a pilot channel that would drain were hurriedly completed and work began. By 2:00 p.m., Monday, February 20, the breach was closed, the pilot channel was carrying Calleguas' flows harmlessly to the ocean, and the masssive cleanup began.

The levee breach caused extended problems to military personnel in base housing. Not until the breach was closed could the majority of these people return to their quarters, and very few families chose to be moved to other quarters while their former quarters were being repaired. The cost of repairing wallboard, flooring, and other major damage to housing structures amounted to $6 million. The value of damaged contents of homes and loss of work added another $3 million to the total loss.

Meanwhile another storm front roared onshore, dumping an additional 2 in. of rain on the already saturated soil. To hear that the southern California coastal area had been declared a disaster area was no surprise to flood fighting forces in Ventura County.

Hydrology

What generated such unusual runoff?

The approximate time of concentration of the Calleguas Creek watershed is four to six hours, depending on storm intensity. One major tributary, the Arroyo Simi, drains nearly 90 square miles of rapidly urbanizing watershed. Simi Valley secondary drains, paved areas, and close-packed roofs all contribute to increased runoff. A recording rain gage in Tapo Canyon, a tributary to Arroyo Simi, showed rain beginning at 5 a.m., February 16. By 10 a.m. the intensity was increasing, with the maximum one-hour rainfall of 0.84 in. occurring between 1500 and 1600 hours. Altogether, 3.89 in. fell between 0500 and 1700 hours.

The three stream gages on Arroyo Simi indicate lag time between rain and runoff as well as travel time. The first peak near the point in the valley where urbanization begins was at 1430 hours. The second, Arroyo Simi at Royal Avenue, peaked at 1530, and the third, Arroyo Simi at Madera Road (the valley mouth), peaked at 1650. The travel time from the mouth of Simi Valley to Point Mugu is at present 2-1/2 to 3 hours through a soft bottom channel subject to large sand waves and lateral erosion.

During the 46 years that the Arroyo Simi has been gaged at the valley outlet, the time from maximum rainfall intensity to peak flow has decreased considerably. The two-hour lag between peak rainfall and peak runoff in February 1980 may indicate the new time of concentration, as the 9,300-cu ft/s peak is the largest in the lengthy record.

Another major lateral, the Arroyo Conejo, drains the Conejo Valley (43 square miles) and Santa Rosa Valley (14 square miles). Although new tracts are efficiently drained with pipes and reinforced concrete channels, several miles of the major drainage channels remain in natural condition to retard and store floodwaters. As a result, even though the drainage area is smaller, the time to peak in Arroyo Conejo is similar to that in Arroyo Simi.

In the February 16 storm the peak discharges from both major tributaries arrived at the stream gage at Camarillo State Hospital at approximately the same time. The 25,300-cu ft/s discharge recorded is the peak of record and approximates a 50-year event using the U.S. Army Corps of Engineers frequency curves for Calleguas Creek's watershed. A current analysis, in view of the significant artificial changes that have taken place during recent years, would decrease the event return.

Sedimentation

Arroyo Simi, Arroyo Conejo Creek, and Calleguas Creek are all natural bottom channels with some local bank protection, stabilizers, and levees along the lower reach as the only improvements. Lateral and vertical erosion during high flows especially contributes to downstream siltation.

From 1969 through 1978 the U.S. Geological Survey monitored sediment discharge on Calleguas Creek near Camarillo State Hospital. According to USGS suspended sediment data for water year 1978 at the Calleguas station 1065 near Camarillo State Hospital (U.S. Geological Survey, 1979), 99 percent of the annual sediment discharge occurred during the three wet months of January, February, and March.

Also, during four days--February 9 and 10 and March 1 and 4, 1978--which is only 1.1 percent of the year, 73 percent of the annual sediment discharge occurred.

By plotting mean daily flow versus suspended sediment discharge (tons per day), a rating curve was developed. This curve was used to estimate the number of tons of sediment that passed by the gage during the 1980 disaster period prior to the levee breach. The rating curve calculations indicate that 1-1/2 million tons of suspended sediment passed the gage between January 9, 1980, and February 16, 1980 (see Figure 3).

If the lower channel trap efficiency experienced in 1978 were repeated, 40 percent of the total suspended sediment could have been clogging the channel. Once breached, not only did the entire sediment load of the stream sweep onto the Oxnard plain, but over 100,000 cu yd eroded by headcutting upstream of the breach was swept out of the channel. Using the same curve, it is possible that 1.7 million tons of sediment poured onto the Oxnard plain before the gap was closed February 20. The sediment was mostly finer material--silt and clay--much of which remained suspended as the flow continued toward the ocean.

Restoration

A recent survey of Calleguas Creek between Highway 1 and Hueneme Road revealed that 260,000 cu yd of sedimentary material will have to be cleaned out of the channel to restore Calleguas Creek to its design capacity.

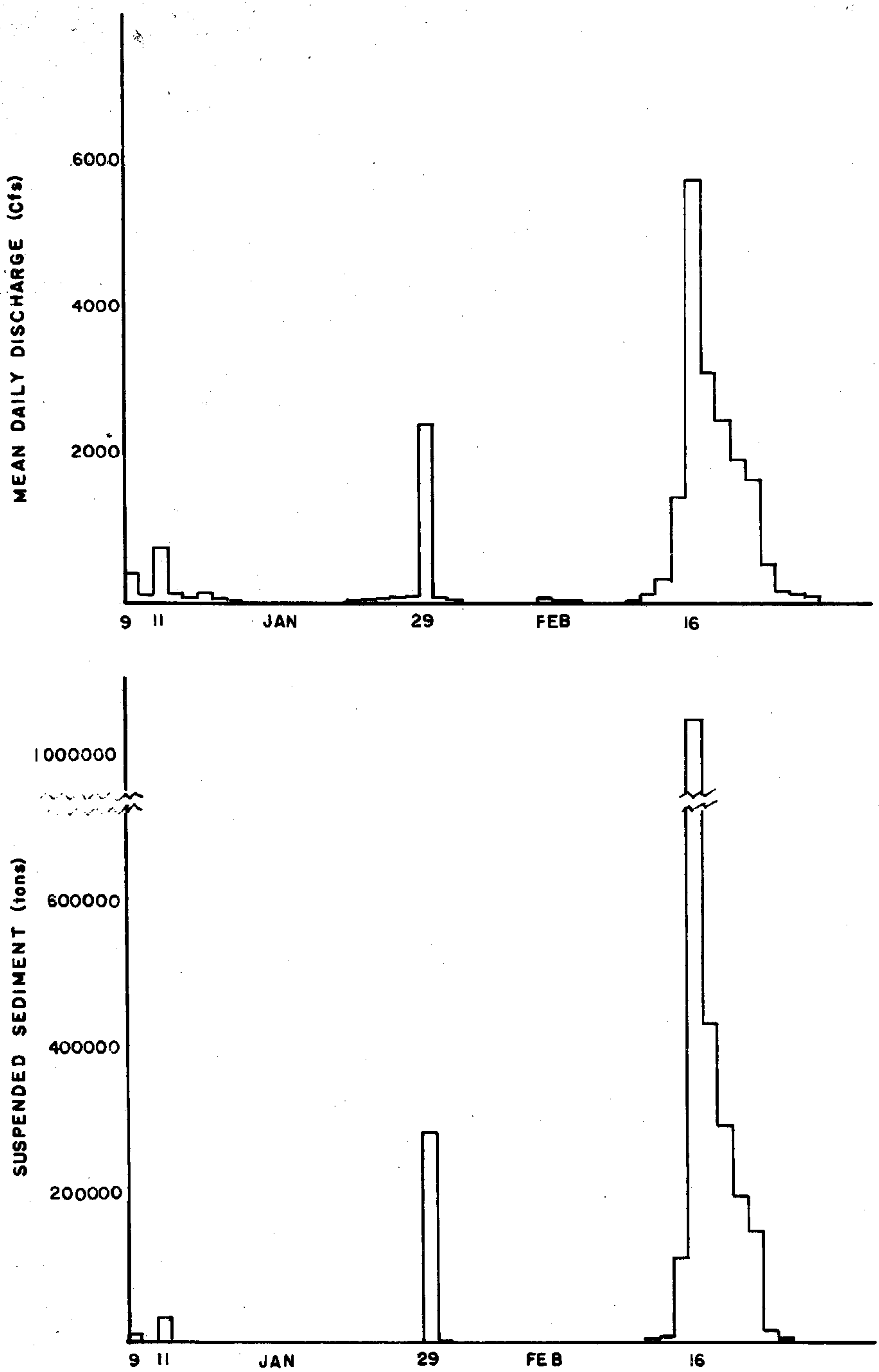


FIGURE 3 Comparison of mean daily discharge to estimated suspended sediment discharge at Calleguas Creek near Camarillo State Hospital for the disaster period January 9, 1980, through February 24, 1980.

With the best of circumstances the contractor cannot begin until mid-October. The cleanout will take approximately 60 days, with a high probability of a storm during that period. At present the estimated channel capacity is less than 8,000 cu ft/s, with a return period of approximately three years. It is only natural then that the personnel at Point Mugu are nervous about flooding. Even with the cleanout, the first significant storm will put the fateful process in motion again by flushing sediments from tributary channels.

Solution

Since an expensive channelization project for Calleguas Creek still lies low on the horizon, a reliable method to warn Point Mugu of impending flooding is being implemented through an unusual three-way agreement between the Navy, the National Weather Service River Forecast Center, and the Ventura County Flood Control District (VCFCD). This flood warning system will be operational by November 1980. There are three major parts to the system.

1. Nine self-reporting rain gages and two stream gages that will transmit real-time data for watershed model calculations
2. A rainfall-runoff model for Calleguas Creek and Revolon Slough
3. Procedures to monitor the changing sediment elevations to provide the best estimate of the capacity of Calleguas Creek below Hueneme Road

The model is being developed by the National Weather Service and will be refined and calibrated as data are gathered.

The equipment was purchased by the Navy and will serve not only the Calleguas flood warning system but also as a backup for the Ventura County system as the two computers are connected with a dedicated phone line. Software was developed by the National Weather Service.

Installation of the self-reporting gages is under way by VCFCD hydrographers. The VCFCD, as an agency of the National Weather Service, will issue flood warnings based upon the input from the elements of the flood warning system. Human judgment will be required to coordinate and interpret three distinct parameters: the quantitative precipitation forecasts from a weather consultant for the following 24-hour period; flood advisories giving predicted discharges for rainfall intensities; and the latest estimate of the channel capacity.

It was necessary to have a means to estimate quickly the average depth of sediment in the channel after each streamflow event. A family of curves was developed showing water surface elevation versus discharge for several incremental depths of sediment in the channel. By plotting the actual discharge determined from the stream gage and the water surface elevation observed at a staff gage located downstream of the stream gage, the average depth of sediment can be estimated.

Readings are taken during and following storms to calibrate the theoretical to the actual amounts. Maintenance of the system is the responsibility of the VCFCD.

There are direct benefits from the flood warning system for all three parties: the Navy will receive 24-hour lead time to warn base personnel to flood-proof or evacuate the base housing; VCFCD will know what is occurring upstream so as to dispatch its small operations group to where it is most needed; and the National Weather Service will receive the real-time data base to assist in flood forecasts for the Southland as the storms move south and east.

FLOOD ASPECTS OF THE CREEK FIRE, SEPTEMBER 1979

As a helicopter approached the burn area several days after the Creek fire finally was out, aerial observers were shocked at the lack of residual vegetation on the 33,000 acres that were blackened. The heat was so intense that 50-year-old chaparral was reduced to powdery white ash.

Ventura County Flood Control, fearful of the consequences of a possible wet year, set about evaluating how big a problem might be facing the area. Calculations of possible postfire debris potential versus the prefire condition were quickly done using formulas developed by Scott and Williams (1974). Storms of various return periods were applied to each of the 13 watersheds, with calculated results indicating that peak flow rates would be 150 to 160 percent greater (Tables 1 and 2).

Two examples of watersheds that bore out the predictions were Canada Larga and Arundell Barranca. Canada Larga includes more than 19 square miles of oak trees, large green chaparral bushes, and oat grass used primarily for grazing beef cattle (see Figure 4). Following each storm from November 2, 1979, checks were made for watershed erosion. Until February VCFCD was fortunate--there had been no high-intensity rainfall. Finally the combination of saturated soil and heavy rain began to move the material in a wave laden with floating and suspended debris. The result for a small home at the mouth of the Canada Larga watershed was total ruin. Further downstream in the floodway is an elementary school. Many volunteers were needed to clean up the unbelievable mess.

Meanwhile, in East Ventura, Arundell Barranca overtopped its west bank for the first time anyone can recall. Nearly half of the drainage area was burned in the steep canyons near the ridge. The 3 square miles of coastal mountains contained in the Arundell Barranca watershed are chiefly undeveloped, with some oil company facilities and a few avocado orchards. At the canyon mouth concrete box culverts convey the flows onto the urbanized plain below. Before the Telegraph Road crossing the channel is a large wooded natural barranca with good storage capacity. East-west Telegraph Road intersects the channel, restricting the flow. On February 16, 1980, at Telegraph Road, the fire-loosened debris, accompanied by suspended silt, broke out and ran to the west in the road's right of way, seeking the ocean. A river of muddy branch-laden water rushed west and south, sweeping Volkswagon buses and stop signs from the streets. In the City Recreation Center at Day Road and Telegraph Road the Ping-Pong tables floated in the muddy water that poured relentlessly through the doors.

TABLE 1 Creek Fire Debris Quantities and Bulking Factor

Location and Channel	Drainage Area (sq miles)	Burned Area (sq miles)	Debris Volumes 100-Yr[a] (cu yd/sq mile)	QB/QC[b] 100-Yr
Flood Zone I				
Skyline	1.12	.22	29,911	1.59
Oak View Drain	.93	.56	43,440	1.66
San Antonio Creek	52.2	7.83	6,609	1.45
Fresno Canyon	1.25	1.24	21,680	1.55
Weldon Canyon	2.13	2.02	30,282	1.61
Canada Larga	19.44	19.44	32,567	1.605
Manuel Canyon	1.08	1.08	20,648	1.54
San Joaquin	1.38	.28	41,160	1.65
Prince-Hall Canyon	5.37	2.42	10,652	1.47
Flood Zone II				
Arundell-Sexton	2.69	1.35	18,030	1.523
Harmon	3.03	2.42	23,630	1.555
Wason	2.65	1.06	15,805	1.51
Ellsworth	7.76	5.43	47,476	1.68

Note: Area total = 101.03 sq miles; burn area = 45.35 sq miles = 29,024 acres, by estimate; actual area = 33,000 acres.

[a]Source: Scott and Williams (1974).

[b]Q Bulked over Q Clearwater.

TABLE 2 Creek Fire Debris Quantities by Scott's Formula

Location	Expected Debris Volumes (cu yd) 10-Yr Storm	50-Yr Storm	100-Yr Storm	Percentage Increase over Prefire Due to Fire
Flood Zone I				
Skyline Drain	12,850	25,800	33,500	207
Oak View Drain	12,900	25,400	40,400	271
San Antonio Creek	132,350	280,450	345,000	217
Fresno Canyon	10,400	20,850	27,100	307
Weldon Canyon	24,000	48,000	64,500	304
Canada Larga	242,800	486,750	633,100	308
Manuel Canyon	8,050	16,800	22,300	308
San Joaquin	20,700	43,150	56,800	207
Prince-Hall Canyon	19,850	43,000	57,200	252
Flood Zone II				
Arundell-Sexton[a]	16,800	36,500	48,500	259
Harmon Barranca[a]	24,800	53,900	71,600	290
Wason	14,606	31,667	42,043	237
Ellsworth-Aliso	127,750	277,341	368,410	282

[a]Analysis by Scott available.

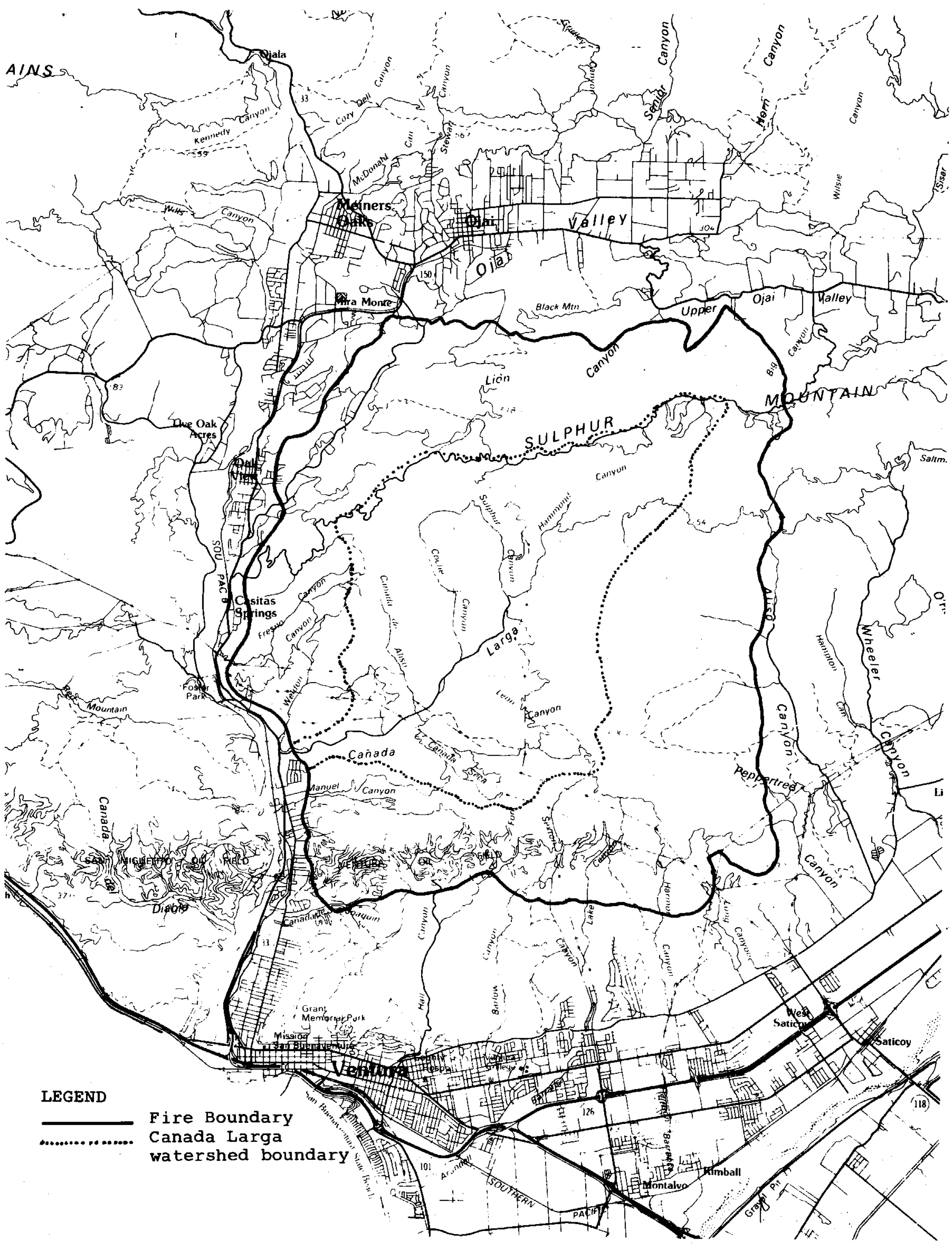


FIGURE 4 Creek fire map showing the extent of the 33,000-acre fire that occurred September 28-30, 1979. Canada Larga, whose drainage area is over 19 square miles, was the largest watershed affected, with 12 others suffering some damage and later related debris flows.

For a brief time Telegraph Road resembled a river. Fifty feet above the curb along the college grounds, debris from the fire and silt settled out as the floodwave receded. The divider median prevented north-south escape flows except at major intersections. Dean Drive runs parallel to the original Arundell Barranca, with many catch basins to intercept and convey the flow to the box culvert under the old streambed.

The effect of a large devastating fire on floodflows was clearly demonstrated in these sample events.

REFERENCES

Scott, K. M., and Williams, R. P. (1974) Erosion and Sediment Yields in Mountain Watersheds of the Transverse Ranges, Ventura and Los Angeles Counties, California, WRI 47-73, U.S. Geological Survey, Menlo Park.

U.S. Army Corps of Engineers (1974) Calleguas Creek--Simi Valley to Moorpark, Ventura County, California, Appendix Plate C-13, U.S. Army Corps of Engineers, Los Angeles.

U.S. Geological Survey (1979) Water Resources Data for California, Vol. 1: Water Year 1978, U.S. Geological Survey, Menlo Park, pp. 471-476.

A CASE STUDY OF A REAL-TIME FLOOD WARNING SYSTEM ON SESPE CREEK, VENTURA COUNTY, CALIFORNIA

by Ira Bartfeld and Dolores B. Taylor

Sespe Creek drains an extremely mountainous, largely undeveloped area in central Ventura County before merging with the Santa Clara River near the city of Fillmore. On March 4, 1978, a major flood on Sespe Creek caused one fatality and over $6 million in damage in the Los Serenos area of Fillmore.

Subsequent to this flood Ventura County, in cooperation with the National Weather Service, used technology developed by the California-Nevada River Forecast Center of the National Weather Service to implement a real-time flood warning system on Sespe Creek.

The value of a local real-time flood warning system in the saving of life and property was quickly realized during the southern California flooding of February 1980. A description of how the system operated in the 1980 high water is compared with the 1978 flood situation.

INTRODUCTION

Death and destruction, as floodwaters ravage a southern California community, has become an all too familiar headline. Dangerous flooding in southern California communities has occurred repeatedly in recent years. Three major Pacific storms occurring in February and March of 1978 and in February of 1980 brought serious flooding to southern California's rapidly responding rivers. The devastation included 38 deaths and close to $400 million in property damage. Floods of this magnitude have occurred repeatedly during this century. This indicates that floods of a far greater magnitude can and probably will occur in the future. There are river basins in southern California where the potential for loss of life is catastrophic. The worst of these is the floodplain of the Santa Ana River, where more than two million people live. What are the alternatives available for minimizing this threat to life and property?

Ira Bartfeld is Senior Hydrologist with the California-Nevada River Forecast Center of the National Weather Service in Sacramento, California. Dolores B. Taylor is Surface Water Hydrologist with the Ventura County Flood Control and Water Resources District in Ventura, California.

Structural solutions can seldom preclude all future flooding. The extremes of nature eventually surpass most design criteria. The resulting disaster may be magnified due to overreliance on structural flood control by floodplain residents. Frequently, even an attempt at structural solutions is not feasible due to physical or economic constraints. Floodplain zoning can reduce future encroachment but rarely offers protection to those who have already occupied the floodplain.

There is another alternative--a relatively inexpensive warning system. This system by itself offers substantial lifesaving potential and is vital for the effective operation of flood control structures. The purpose of this paper is to demonstrate the impact of a modern automated flood warning system on a community with a serious flood problem. The initial portion of the paper discusses the roll of the National Weather Service in a cooperative automated flood warning system. The final portion traces the implementation and operation of the flood warning system on Sespe Creek, Ventura County, California. It focuses on its social and economic impact on the City of Fillmore during the high water of February 1980.

ROLE OF THE NATIONAL WEATHER SERVICE

The National Weather Service, through its California-Nevada River Forecast Center in Sacramento, is implementing automated flood warning systems in California. These systems are being implemented in cooperation with local agencies in areas where there are serious flood problems. A brief discussion of the rationale and components of the cooperative automated flood warning system is necessary in order to understand fully the role of the National Weather Service as well as that of the cooperating agencies.

Effective flood warnings providing the maximum response time require real-time hydrologic analysis of changes in field storm conditions. Networks of self-contained, self-powered event-reporting gages transmit via radio, to a specially configured data acquisition system, precipitation amounts as the precipitation occurs (Burnash and Twedt, 1978). The acquisition system consists of a radio receiver, a minicomputer system, and a display screen located at the local cooperating agency. Precipitation information is acquired and displayed in a continuous mode. The River Forecast Center, which is linked by phone to the local minicomputer, obtains and analyzes these data using a streamflow simulation system calibrated for the individual basin (Burnash et al., 1973). Peak streamflow advisories based on these simulations are updated and transmitted to the local minicomputer as required to define the flood potential. The object is to make available at the local level the information required for maximizing effective flood warning lead time. Local officials make decisions on flood response actions based on information generated by the flood warning system and a locally developed flood response plan.

The National Weather Service contributes substantial resources to a cooperative flood warning system in the following areas.

1. System design, including hardware configuration and sensor requirements and location

2. Software package for collecting, processing, and displaying data and advisories
3. Calibration of streamflow simulation models for the basin
4. Continuing peak streamflow advisory service based on the streamflow simulation
5. Continual updating and recalibration of streamflow simulation models as required
6. Training in system installation and operation

The cooperating agency provides and maintains locally installed hardware, ensures development and operation of a flood response plan, monitors the system, and, based on National Weather Service guidance, issues local flood warnings as appropriate.

This information on flood warning systems and the roll of the National Weather Service provides the background for the study of the implementation and operation of such a system on Sespe Creek and its consequent benefits to the city of Fillmore.

SESPE CREEK STUDY

Sespe Creek rises in extremely mountainous terrain northwest of Ojai in Santa Barbara County. The creek flows 20 miles in an easterly direction and then about 10 miles south, merging with the Santa Clara River near Fillmore. More than 90 percent of its 270-square-mile drainage area is undeveloped U.S. Forest Service property, including a large portion of the Sespe Condor Refuge.

The area prone to extensive flood damage is along the lower 2 miles of river channel. This area includes residential tracts in the City of Fillmore, commercial and industrial areas, and many orchards. The area has a history of flooding due primarily to prolonged periods of heavy precipitation during the winter months.

Flooding of 1978

During the storms of February 5-11, 1978, the peak flow of record, 73,000 cu ft/s, occurred at Fillmore. There were partial evacuations and much confusion in response to rapidly rising creek levels. Overbank flow was negligible, but more than 10 ft of sediment was deposited in the channel along the lower reach. On March 4, 1978, a peak flow of 54,000 cu ft/s broke out of the sediment-laden channel, causing a major flood in Fillmore. Tons of silt and debris were carried into over 370 homes. There was one fatality and structural damage to 200 homes from floodwaters. Total damages to the Los Serenas area of Fillmore were estimated to be $6.2 million (Figures 1, 2, and 3). Frantic efforts of individual homeowners to save what they could during the flood impeded evacuations, blocked emergency equipment and flood fighting equipment, and severely hampered flood fighting efforts.

Implementation of the Sespe Creek Flood Warning System

As a result of the flooding of March 1978, the Ventura County Flood Control District, in cooperation with the California-Nevada River Forecast

FIGURE 1 Flooding in Fillmore, March 1978.

FIGURE 2 Residential flood damage in Fillmore, March 1978.

Center of the National Weather Service, began implementing an automated flood warning system on Sespe Creek. By March 1979 six rain gages and a radio repeater installation were in place in the Sespe Creek watershed (Figures 4, 5, and 6). The minicomputer was in place at the offices of the Ventura County Flood Control District (Figure 7), receiving and displaying data from the remote rain gages, and peak flow advisories from the River Forecast Center, based on calibration of the streamflow simulation model for Sespe Creek, were being sent to the minicomputer.

The City of Fillmore then instituted a three-phase plan for flood response. As part of this plan, Ventura County Flood Control District officials monitor the flood warning system and issue bulletins that trigger the appropriate phase of the plan.

Storm events producing minor to moderate stream levels occurred in March 1979 and January 1980. These events provided county officials with some initial experience in working with the warning system. In addition, they provided valuable data to River Forecast Center hydrologists for recalibration of precipitation inputs to the simulation model.

Storms of February 1980

On February 13, 1980, the first in a series of Pacific storms moved across Ventura County. On Saturday morning, February 16, the biggest storm of the series approached the area. Figure 8 shows the sequence of peak flow advisories transmitted from the River Forecast Center computer in Sacramento to the Ventura County minicomputer on the morning of the sixteenth. Ventura County had a forecast from their contract meteorologist of 3 in. of rain in the mountains after 10 a.m. The predicted peak flow for this amount of rainfall was about 34,000 cu ft/s. The capacity of Sespe Creek near Fillmore due to siltation from the previous storms was now reduced to approximately 30,000 cu ft/s. These facts indicated to the Ventura County Flood Control District that the city of Fillmore was faced with the possibility of breakout and a repeat of the 1978 flooding. As a consequence, phase I of their response plan was instituted. At 11:15 a.m. police and firemen were placed on stand-by. Residents were notified door to door in designated areas of the possibility of flooding. Information on survival, movement of vehicles, and property protection measures were given to residents as part of the preparation for possible flooding.

Flood Control District personnel monitoring incoming precipitation reports advised officials in Fillmore of the increasing flood threat. Figure 9 is a copy of one of the actual data base displays available to district personnel on a continuous basis. Fillmore instituted phase II of their response plan at 11:55 a.m. Emergency centers, personnel, and communications were activated. Residents in flood-prone areas were again alerted and a voluntary evacuation program was initiated. A bulldozer was positioned along the bank at the weakest point of the channel, above the Los Serenos area of Fillmore.

As precipitation amounts approached 3 in., phase III of the Fillmore response plan was initiated at 3:15 p.m. People in designated areas were

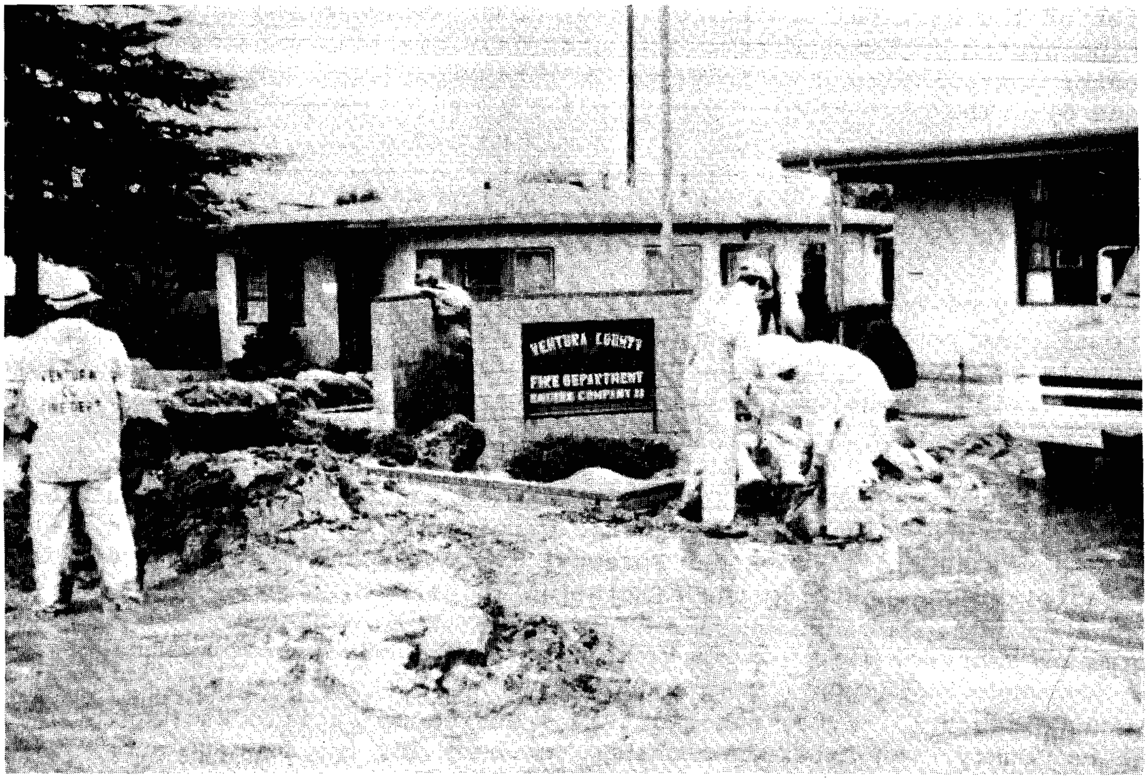


FIGURE 3 Flooding in Fillmore, March 1978.

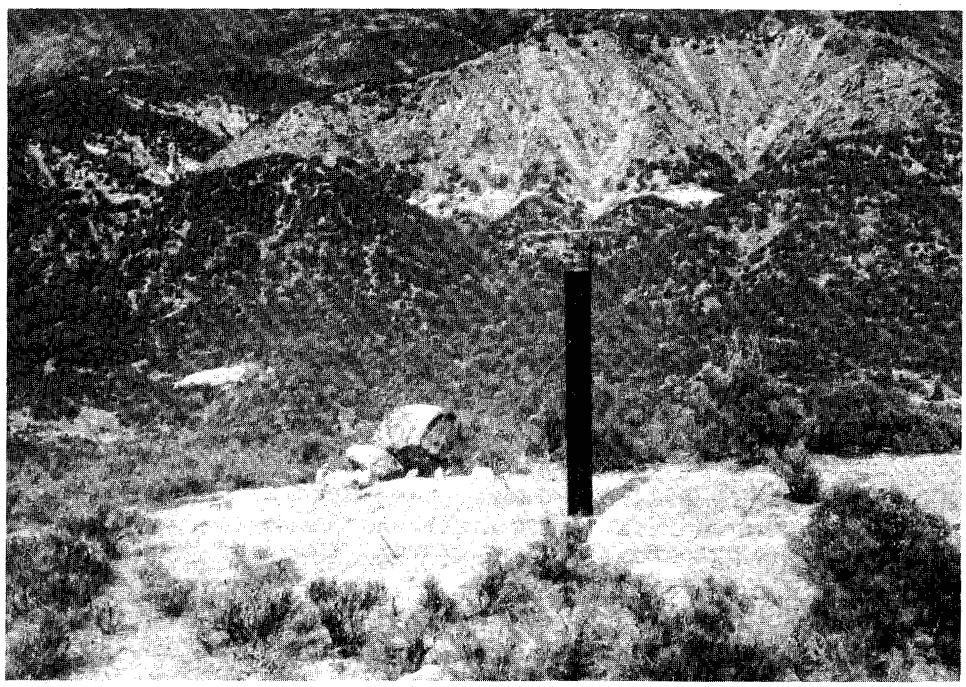


FIGURE 4 Remote self-reporting rain gage.

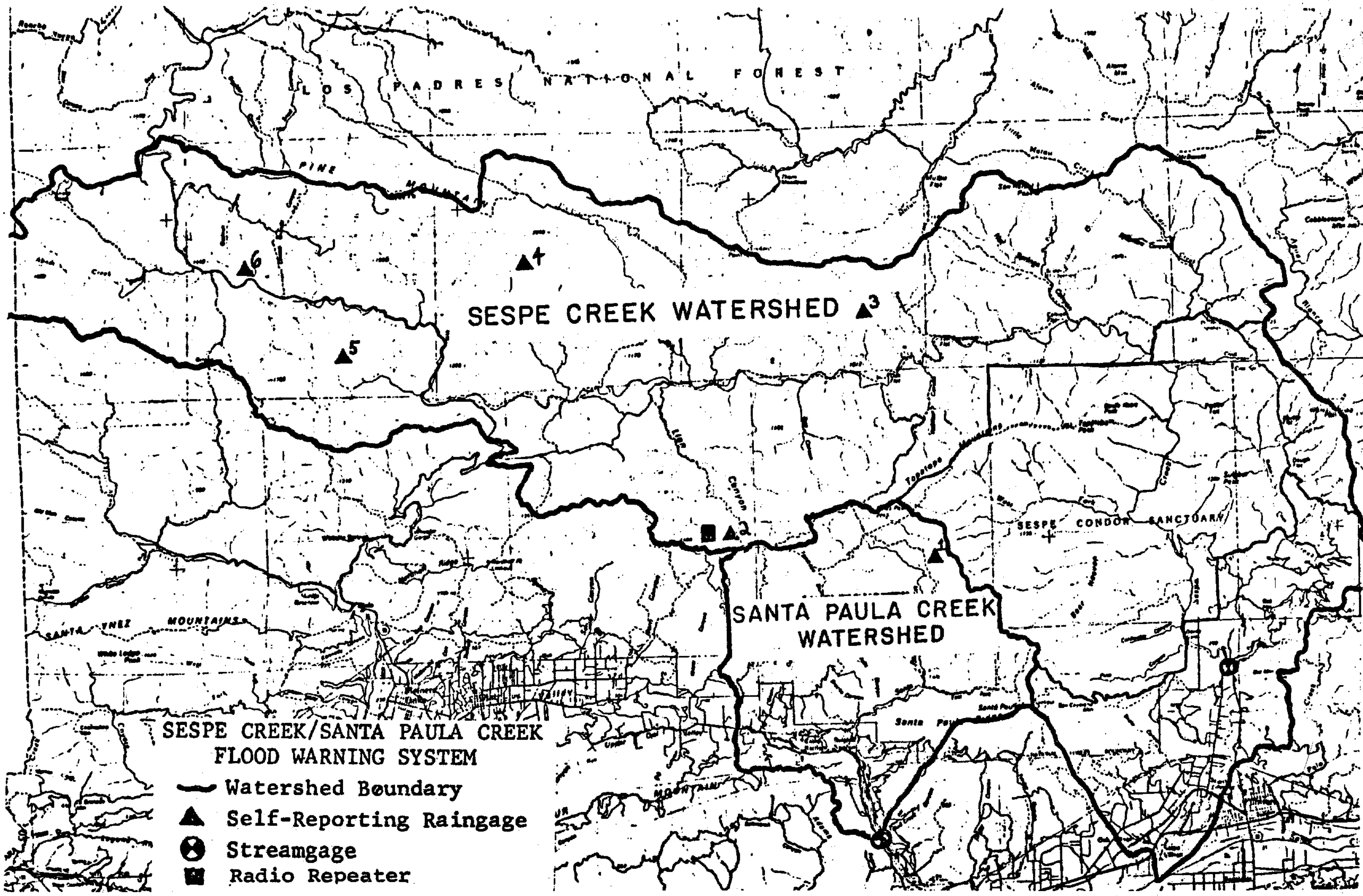


FIGURE 5 Sespe Creek reporting network.

FIGURE 6 Sisar Peak radio repeater.

FIGURE 7 Ventura County minicomputer.

V E N T U R A   C O U N T Y   F L O O D   A D V I S O R Y

Issued at 558 Hours on 2/16/80

Forecast Peak Flows in Thousand CFS Resulting from Precipitation

Falling After 400 on 2/16:

Precipitation in Inches	Sespe CR NR Fillmore	Precipitation in Inches	Santa Paula CR NR Santa Paula
1	7.02	1	1.06
2	15.84	2	2.40
3	24.83	3	3.76
4	33.85	4	5.13
5	42.89	5	6.50

Issued at 1023 Hours on 2/16/80

Forecast Peak Flows in Thousand CFS Resulting from Precipitation

Falling After 1000 on 2/16:

Precipitation in Inches	SESPE CR NR Fillmore	Precipitation in Inches	Santa Paula CR NR Santa Paula
1	16.17	1	2.45
2	24.97	2	3.78
3	33.99	3	5.15
4	43.04	4	6.52
5	52.09	5	7.89

FIGURE 8 Peak flow advisories sent from the River Forecast Center to the Ventura County minicomputer.

Federal-State Hydrologic Data Collection System ** Sespe Creek

	Sta 10	Sta 20	Sta 30	Sta 40	Sta 50	Sta 60
Date, Time, Value of Last Transmit	2/16 1622 88	2/16 1625 43	2/16 1628 68	2/16 1621 69	2/16 1641 64	2/16 1652 44
Precipitation						
Last 10 min	0.00	0.00	0.00	0.00	0.00	0.04
Last 30 min	0.00	0.00	0.04	0.00	0.00	0.08
Last 60 min	0.28	0.20	0.16	0.12	0.12	0.16
For 6 Hours Ending at 1600	3.56	3.72	3.12	2.48	3.84	3.28
For 24 Hours Ending at 1600	4.84	5.36	4.36	3.52	5.64	5.04
Since 0800 Today	4.44	4.72	3.92	3.00	4.84	4.24
Since 1600	0.24	0.16	0.16	0.08	0.12	0.12
1500 thru 1600	0.72	0.64	0.60	0.44	0.48	0.48
1400 thru 1500	0.80	0.68	0.44	0.32	0.64	0.60
1300 thru 1400	0.44	0.68	0.64	0.40	0.80	0.60
1200 thru 1300	0.36	0.44	0.44	0.44	0.72	0.44
1100 thru 1200	0.68	0.64	0.48	0.40	0.52	0.56
Since 1600						
1000 thru 1600	3.56	3.72	3.12	2.48	3.84	3.28
400 thru 1000	1.12	1.56	1.24	0.96	1.64	1.72
2200 thru 400	0.00	0.00	0.00	0.00	0.04	0.04
1600 thru 2200	0.16	0.08	0.00	0.08	0.12	0.00
1000 thru 1600	0.28	0.04	0.08	0.12	0.12	0.08
400 thru 1000	0.40	0.32	0.40	0.28	0.40	0.32
2200 thru 400	0.40	0.50	0.28	0.84	0.72	0.48
1600 thru 2200	0.72	0.52	0.24	0.52	0.52	0.40

FIGURE 9 Example of a data base display available to district personnel.

evacuated and flood fighting equipment was readied. As Sespe Creek approached its crest of 36,000 cu ft/s at 5:15 p.m., it began to break out of its banks above the Los Serenos tract. The bulldozer in position at that point was able to berm up the bank, preventing any further breakout. The heavy rain ended and the creek began receding with no flood damage to the City of Fillmore.

Warning System Benefits

Ventura County Flood Control District officials and Fillmore city officials credit the flood warning system and response plan with preventing a repeat of the 1978 flooding. The lead time available to the City of Fillmore largely eliminated the potential for loss of life during the February 1980 high water. The capability of the system to generate discrete site-specific warning generated a response that prevented flood damage to over 200 homes. Through the investment of about $50,000 in a flood warning system, total flood damages of over $5 million were likely prevented in Fillmore. This was accomplished in an orderly manner with a minimum of unnecessary disruption to floodplain residents. The contrast between this event and the disorganization of affected residents during 1978 is yet another example of the benefits of the warning system.

The benefits of the flood warning system are dramatically brought home when this high water event is compared with the flooding on nearby Calleguas Creek. A record crest of approximately 24,000 cu ft/s broke out of the Calleguas Creek levees on February 16, without warning, flooding the Point Mugu Naval Base housing area. Residents had to be evacuated in waist-deep water, and personal property damage was estimated at $5 to $8 million. It was fortunate that there were no fatalities. A flood warning system similar to the one on Sespe Creek could have reduced property damage and precluded the threat to life. It might have prevented much of the damage by providing the lead time warning for positioning heavy equipment to reinforce the levee.

As a result of the Sespe Creek and Calleguas Creek flooding experiences of 1980, the Navy realized the need for a flood warning system. The Pacific Missile Test Center at Point Mugu, the National Weather Service River Forecast Center at Sacramento, and the Ventura County Flood Control District are cooperatively implementing a flood warning system for Calleguas Creek that will be operational during the winter of 1980-81.

CONCLUSION

The case study of the Sespe Creek flood warning system demonstrates the life- and property-saving impact of a real-time cooperative flood warning system. This type of system is especially appropriate to the flood problems of southern California's rivers and creeks. It should be considered as a major nonstructural flood control alternative and as a necessary adjunct to flood control structures.

REFERENCES

Burnash, R. J. C., and Twedt, T. M. (1978) Event Reporting Instrumentation for Flash Flood Warnings, preprint volumes, Conference on Flash Floods, American Meteorological Society, Boston.

Burnash, R. J. C., Ferral, R. L., and McGuire, R. A. (1973) A Generalized

Streamflow Simulation System, National Weather Service and California Department of Water Resources, Sacramento.

Ventura County Flood Control District (1980) Report on Sespe Creek Warning System, Ventura County Flood Control District, Ventura, California.

Wahl, K. L., Crippen, J. R., and Knott, J. M. (1980) Floods of January and February 1980 in California, Open File Report 80-1005, U.S. Geological Survey, Menlo Park, California.

THE SANTA ANA RIVER IN ORANGE COUNTY, CALIFORNIA: A CASE HISTORY IN SEDIMENT TRANSPORT

by Carl R. Nelson

The Prado Dam, which has regulated peak discharges on the Santa Ana River since 1941, substantially reduced floodflows from the storms of 1969, 1978, and 1980. However, the sustained flows of 1969 caused serious lateral erosion and threatened to break out into the western coastal plain of Orange County. As a result, the Orange County Flood Control District began a program of channel improvements in 1969 to upgrade sideslope protection and incorporate additional grade reduction structures and water diversion works. These systems worked well in the storms of 1978 and 1980 with one exception: sediment starvation caused significant headward scour along a reach where the drop structures had not been fully completed. This demonstrates the problems that less than peak flows can cause in rivers where bank stabilization has removed previous sources of sediment.

INTRODUCTION

The Santa Ana River watershed envelops approximately 2,200 square miles. The river begins at elevations above 10,000 ft in the San Bernardino Mountains, flows across portions of the counties of San Bernardino and Riverside, and enters Orange County in the narrow Santa Ana Canyon, as illustrated in Figure 1. In its passage across the alluvial coastal plain of Orange County, it runs from the mouth of Santa Ana Canyon near Yorba Linda to an ocean entry at Newport Beach, a distance of approximately 20 miles (see the profile in Figure 2). Since 1941 peak discharges have been regulated by Prado Dam near Corona, California.

Large floods are known to have occurred in the years 1825, 1861, 1884, 1916, and 1938, as indicated in Figure 3 (which also shows the rainfall history). For an unknown time prior to 1825 the river's course across the coastal plain of Orange County was westerly to an ocean entry near Seal Beach. During the 1825 flood the river's course was changed by alluvial accretion to an ocean entry at Newport Beach, approximately 10 miles downcoast. Since that time, man's efforts to confine floodflows to a single channel have been directed toward this latter ocean entry.

Carl R. Nelson is Director of Public Works with the Orange County Environmental Management Agency in Santa Ana, California.

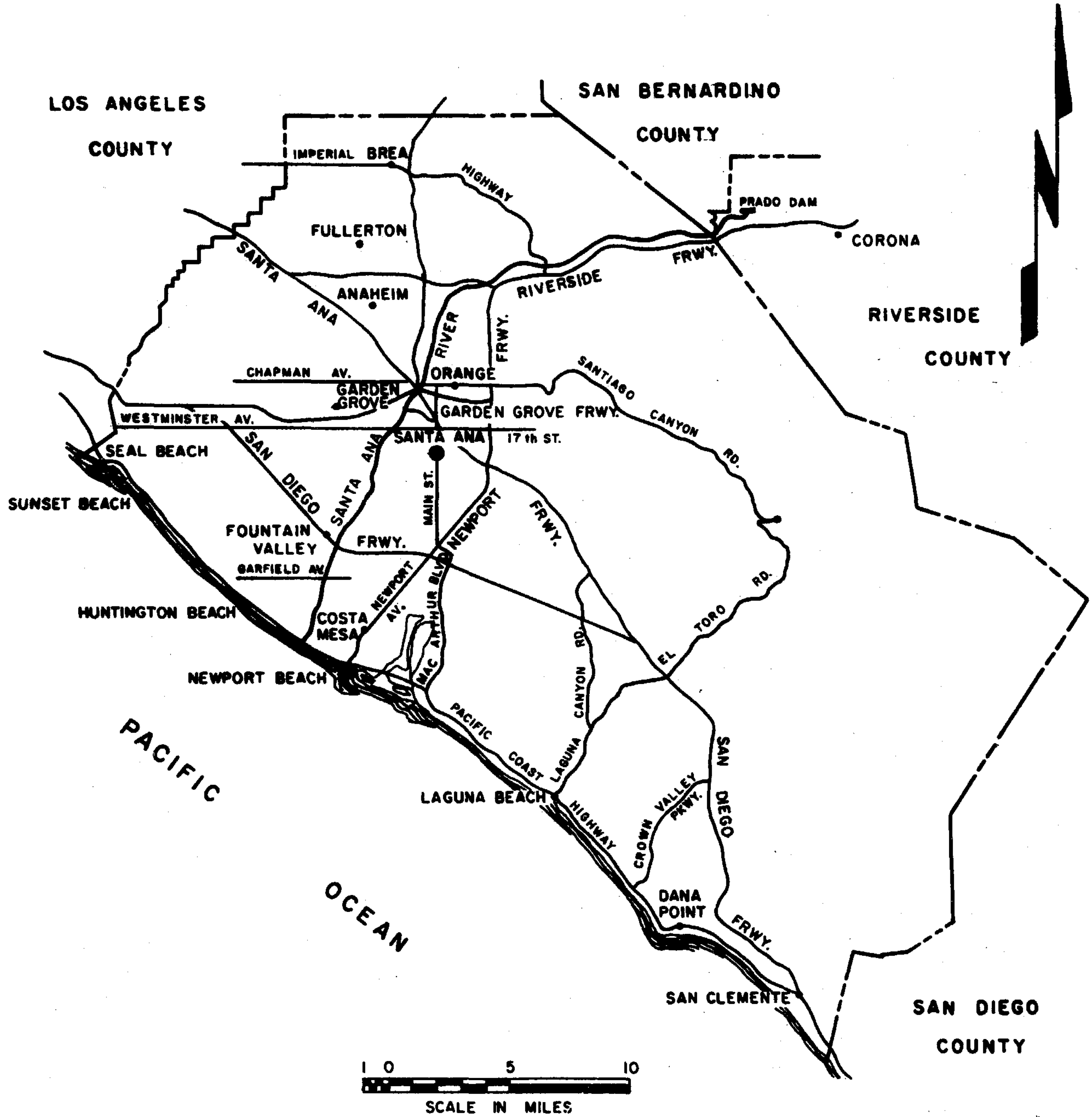


FIGURE 1 Map of Orange County, California, showing Santa Ana River.

The most significant of these efforts has been the completion of Prado Dam, with a storage capacity of 215,000 acre-ft, in 1941 by the U.S. Army Corps of Engineers at the head of Santa Ana Canyon near Corona. Large floodflows were substantially reduced in 1969, 1978, and 1980 by the existence of Prado Dam (see Figure 4 for the 1980 hydrograph). Although designed for a peak discharge of 9,300 cu ft/s, the Corps limited the discharge in each of

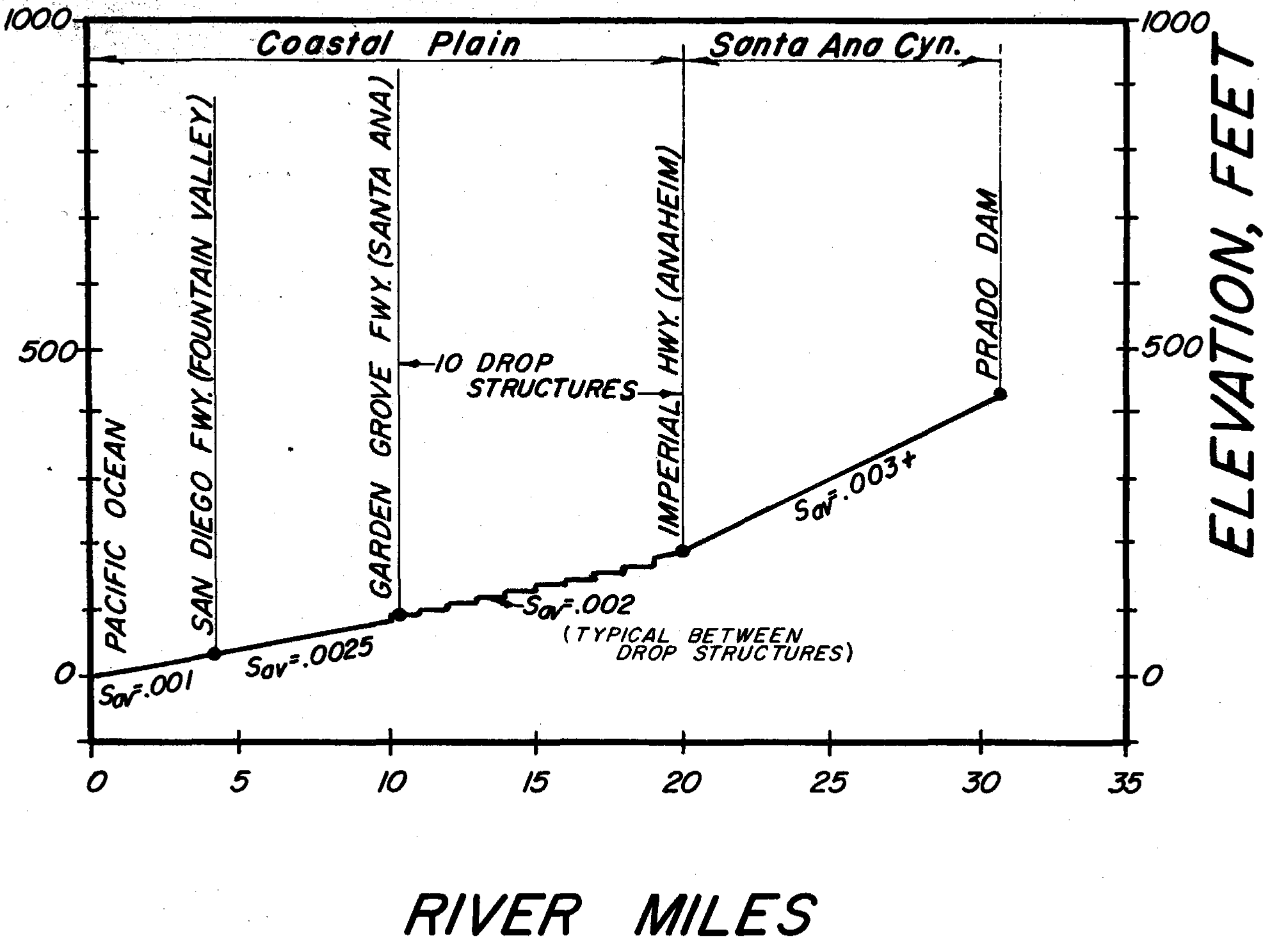


FIGURE 2 Santa Ana River profile from the Pacific Ocean to Prado Dam.

these three recent floods to not more than 6,000 cu ft/s because of serious erosion problems, both in the unimproved Santa Ana Canyon area and in the improved channel between Imperial Highway and the ocean. In and near the ocean, large volumes of sediment accumulated, reducing the channel's capacity for subsequent floodflows.

FLOOD HISTORY

The farmers of Orange County first channelized the Santa Ana River to capture its perennial flow for irrigation. Subsequently, for protection against infrequent but large floods, earthen levees were constructed and revetted with pipe and wire; but these levees gave only a false sense of security, typically failing during very large floods.

The flood of the century occurred in 1938, with a peak flow of 100,000 cu ft/s in Santa Ana Canyon. This great flood was considered to be a 1-in-40-year event, and the peak discharge to the ocean was approximately

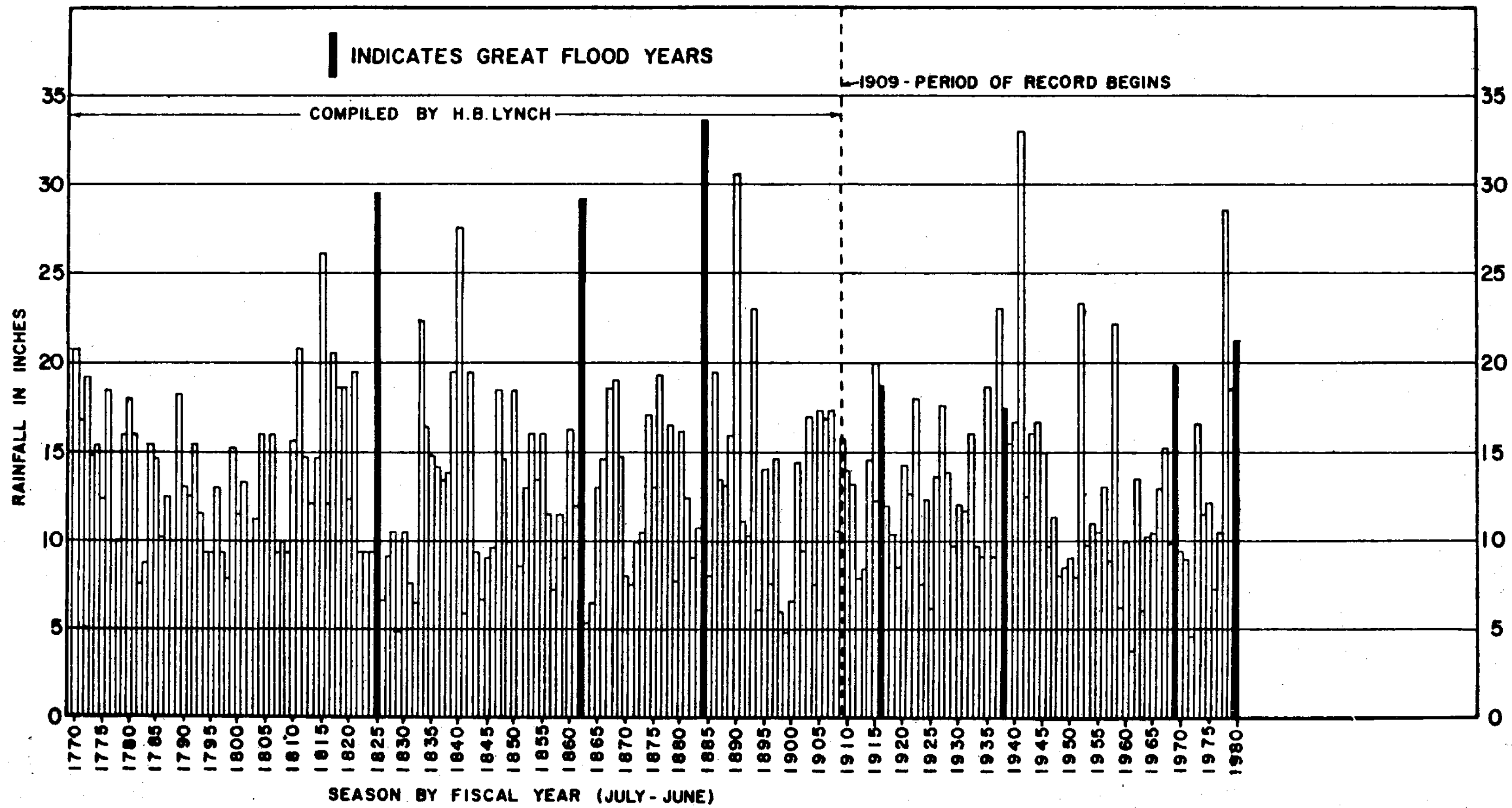


FIGURE 3 Flood years and yearly rainfall at Santa Ana since 1769.

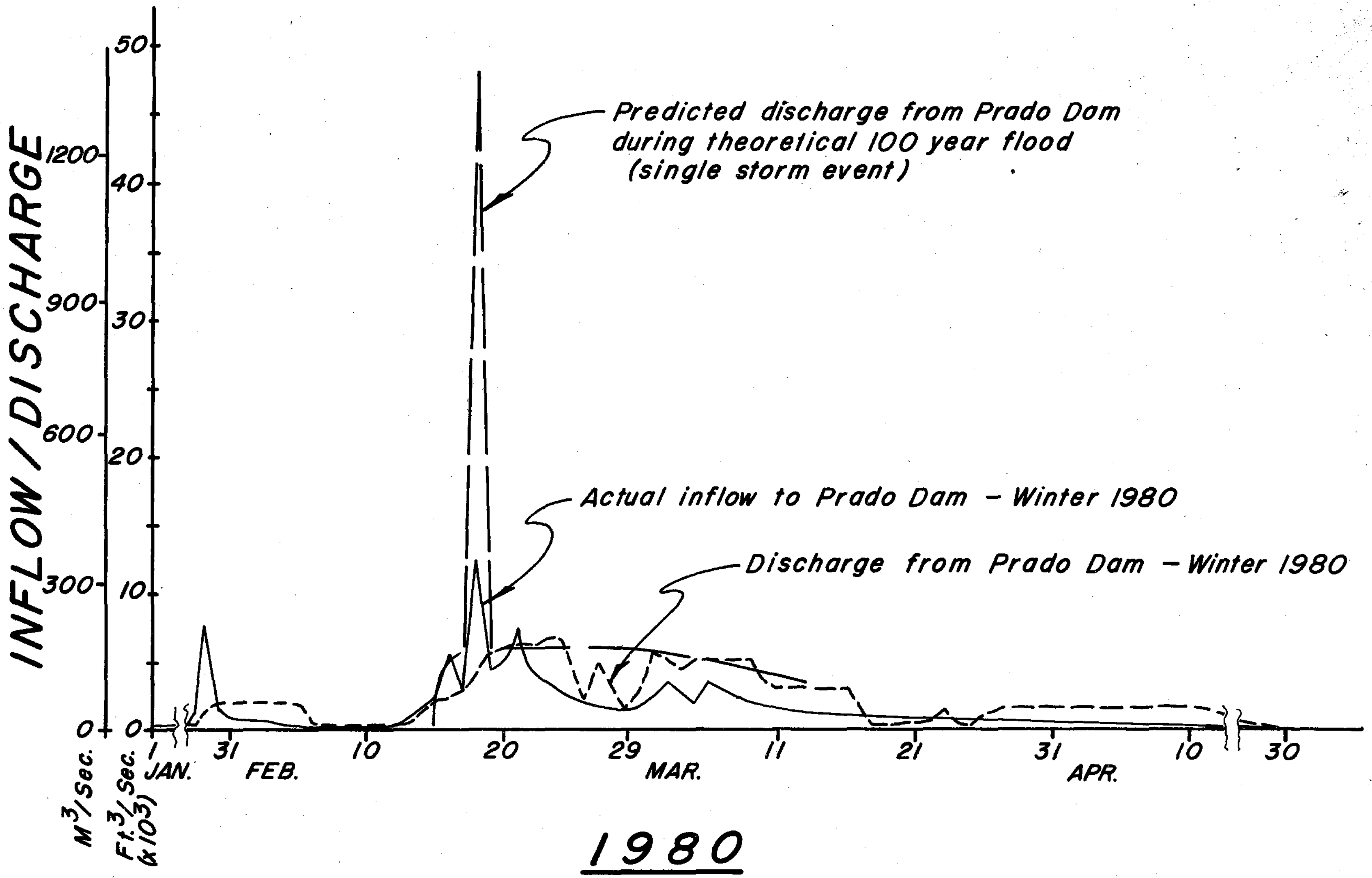


FIGURE 4 Discharge from Prado Dam during the winter of 1980.

45,000 cu ft/s, the remainder of the peak having been attenuated in spreading across the coastal plain.

Prior to 1969 the peak discharge from Prado Dam had been approximately 2,000 cu ft/s, which was contained within the channelized portion of the river with little consequence. The channel had been improved by the Orange County Flood Control District in 1956 by the addition of an asphalt sideslope lining to the formerly unprotected levees. The channel invert in the lined section between Pacific Coast Highway and Seventeenth Street remained a sand bottom in the interest of economy. Between Seventeenth Street and Imperial Highway a pipe and wire-type protection had been installed following the 1938 flood, and this protection had been untested until the 1969 floods.

Two floods occurred in 1969, both delivering peak flows at Prado Dam approximating 75,000 cu ft/s and both considered approximately 1-in-30-year events. The floods of 1978 and 1980 were very damaging, but not exceptionally severe in terms of peak flow. The peak discharges from Prado Dam in 1969 and 1980 were approximately 5,000 to 6,000 cu ft/s, and in each event the water storage at Prado Dam accumulated to a total of approximately 125,000 acre-ft; thus evacuation of the water in storage required sustained discharges lasting several weeks.

Prior to 1969 a large percentage of the annual storm runoff in the Santa Ana River had been conserved by spreading in the sandy bed of the river between Imperial Highway and Katella Avenue. Lateral erosion had been of relatively small consequence since construction of Prado Dam. The higher sustained flows of 1969 (5,000 cu ft/s), however, caused serious lateral erosion, and the river threatened to break out across Anaheim, as had occurred with the much larger flood of 1938. Thus, in 1969, the Orange County Flood Control District began a program of channel improvements that included rock-protected sideslopes in the permeable water-spreading area and grade reduction structures and diversion works for off-channel water spreading.

IMPROVEMENT HISTORY

Early in the twentieth century, farmers who had been adversely affected by the meandering of the river organized stormwater protection districts. After the floods of 1938 demolished their earthen levees, a unification effort was commenced whereby the separate stormwater protection districts would convey to the Orange County Flood Control District the levee improvements and the rights-of-way. It was not until the Flood Control District's 1956 bond election was passed, however, that a comprehensive effort could be undertaken to improve the levees further. Since that time the levee system has undergone the following evolution.

1. The Flood Control District accepted ownership of the stormwater protection districts' easements and the responsibility for maintaining the system. The then-existing protective works were no more than pipe and wire revetment between Seventeenth Street in Santa Ana and Imperial Highway, the reach where lateral erosion was most prevalent. Between Seventeenth Street and the ocean the channel gradient was flatter, and the only protection

against lateral erosion was in vegetative cover that had been nurtured on the levees by the old stormwater protection districts.

2. Although the improvements to the river, as just described, had sufficed reasonably well during the dry cycle of the late 1940s and 1950s, the value of improved property across the coastal plain had increased dramatically between 1950 and 1960; the county's population had grown from 216,224 to 703,925. During that period the maximum discharge that had been released from Prado Dam was only 2,000 cu ft/s. Vertical erosion had never been a problem during that period; lateral erosion had been manageable with a modest force of personnel and equipment operated by the Orange County Flood Control District.

3. Under the 1956 bond issue the Flood Control District improved the channel with rock-revetted sideslope from the ocean to approximately Garfield Avenue. From Garfield to Seventeenth Street the sideslopes were improved with a wire-mesh-reinforced asphalt-concrete sideslope revetment. Due to an impermeable soil layer overlying the groundwater basin in this reach, water conservation was not an issue. For economy the channel invert remained earthen.

4. In the early 1960s, between Seventeenth Street and Imperial Highway, intermittent improvements, financed from the district's annual property tax levy, were installed where dictated by either failures of the pipe and wire revetment or where improvements were made in conjunction with the state freeway project or county arterial highway projects.

5. In 1964 the Board of Supervisors, recognizing that population growth in west Orange County depended on the foregoing improvements for flood protection, commissioned a study of the work that would be necessary to stabilize further the Santa Ana River levees between Katella Avenue and Imperial Highway. This was in conjuction with channelization that would improve water-spreading capabilities in the wide sandy bed of the river.

6. The consultant's report (Leeds et al., 1964) recommended a dual-channel concept whereby a primary floodway would be constructed within the wide riverbed. It would have rock-revetted sideslopes and grade reduction structures. Gated outlet structures would release floodwaters into parallel off-channel spreading grounds to be operated by the Orange County Water District. The consultant recognized the necessity to cope with sediment transport and recommended that a sediment transport equilibrium be established. Bed load eroded from the unimproved Santa Ana Canyon area would be transported through the grade-reduced channel and then enter a lined channel section, within which it would flow along the natural gradient from Seventeenth Street to the ocean. Empirically, it was anticipated that sediment conveyed into the asphalt-lined levee section would eventually reach the coastline and thus replenish the sandy beaches of the area. Prior to the floods of 1969 this had been the Flood Control District's experience.

The two large floods of 1969 produced peak flows of 75,000 cu ft/s into Prado Dam, along with record high water levels. According to the Corps of Engineers' operations plan, the gated discharge should have been gradually increased to 9,300 cu ft/s. However, with a discharge of approximately 5,000 cu ft/s, plus side inflow, a peak discharge in the lower Santa Ana River of approximately 19,000 cu ft/s was recorded at the Fifth Street gaging station in Santa Ana. The Corps of Engineers, therefore, restricted gated releases to the rate of 5,000 cu ft/s, which, although extremely damaging by way of lateral erosion to the pipe and wire revetment, was successfully contained by the flood fighting efforts of the Orange County Flood Control District and the Corps of Engineers operating under Public Law 99.

By the time the reservoir had been emptied after the February 1969 flood, damages to the pipe and wire revetment and the asphalt-concrete channel lining were extensive. Furthermore, a deposit of approximately one million cubic yards of sediment choked the mouth of the Santa Ana River, and a delta of unmeasured quantity was deposited in the offshore littoral zone. Clearly, there was a need to accelerate implementation of the dual-channel concept, to avoid a disastrous future breakout of the Santa Ana River across the heavily populated west Orange County coastal plain.

Between 1969 and 1978 the following improvements were installed along the Santa Ana River.

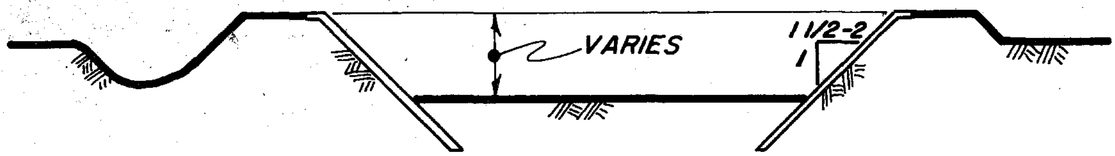
1. From the mouth of Santa Ana Canyon, above Imperial Highway, to the Garden Grove Freeway, upstream from Seventeenth Street, the primary floodway was improved with rock-revetted sideslopes (see Figure 5b). Nine of the consultant-recommended drop structures (Figure 6) had already been installed, along with the water conservation features between Imperial Highway and Katella Avenue. Funds had not yet become available to complete all of the grade stabilization structures; these had been deferred in locations that appeared least vulnerable to vertical erosion.

2. Between Garden Grove Freeway and Seventeenth Street no further improvements had yet been made, in that the pipe and wire protection had performed admirably with the comparatively cohesive earthen materials of the streambed and banks.

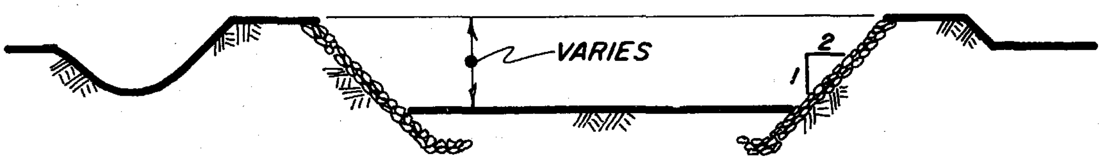
3. The asphalt-concrete sideslope paving between Seventeenth Street and Garfield had been totally replaced with a reinforced-concrete sideslope paving (see Figure 5a). This sideslope protection has a depth below invert of 7 ft and performed very well under the severe test of the winter of 1977-78. However, evidence of sediment transport nonequilibrium was revealed in the failure of the drop structure furthest downstream, below Katella Avenue, due to a deficiency of bed load and attendant piping under the structure's foundation.

4. The drop structure was replaced during the summer of 1979, and four of the previously deferred structures were installed between the failed structure and Seventeenth Street.

5. The reinforced-concrete sideslope paving project was completed from Garfield Avenue to the Pacific Coast Highway during the summer of 1979.



A. CONCRETE SIDESLOPES



B. ROCK-REVETTED SIDESLOPES

FIGURE 5 Typical improved sections of the Santa Ana River (sand bed).

SEDIMENT TRANSPORT PROBLEMS

The flood of 1978 was of serious magnitude, again creating a sustained discharge from Prado Dam on the order of 2,000 cu ft/s. This discharge was easily contained within the grade-stabilized channel improvements, with one significant exception: the drop structure system had not been completed in the reach between Orangewood Avenue and Seventeenth Street in Santa Ana. The consequence was headward scour, which led to structural failure of the last downstream drop structure. Under emergency authorization the damaged drop structure was removed and replaced with an improved cutoff wall, and four additional grade stabilization structures were completed between Orangewood Avenue and Seventeenth Street.

Not enough funds were available to backfill the new drop structures so as to replace the streambed material lost in the headward scour. The moderately wet winter of 1978-79 did not produce enough sediment to backfill the new drop structures; hence the stage was set in the winter of 1979-80 for sediment starvation in the soft-bottom concrete-sideslope channel section running downstream from Seventeenth Street.

The winter of 1979-80 was the third consecutive winter of above-average rainfall in the Santa Ana River watershed (Figure 3). The steep, unimproved channel reach from Prado Dam to Imperial Highway produces erodible velocities

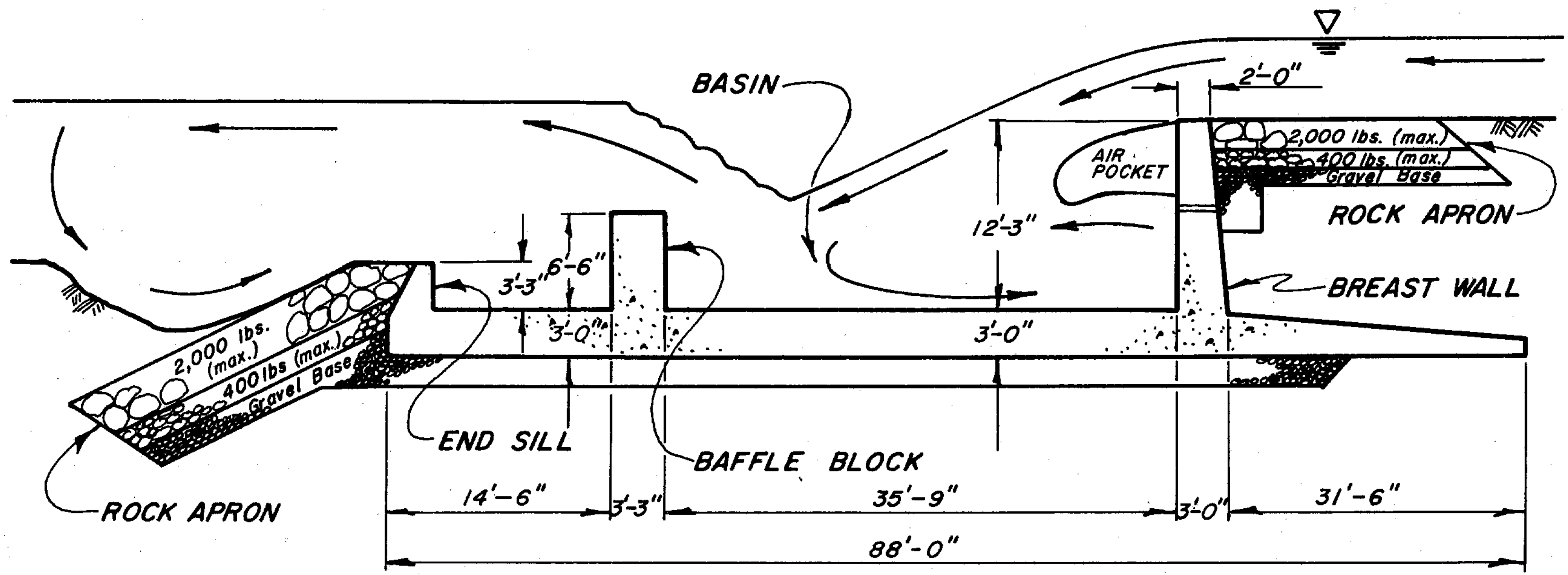


FIGURE 6 Drop structure on the Santa Ana River.

(averaging approximately 13 ft/s) and thus is the source of sediment needed to establish equilibrium of sediment transport in the grade-stabilized channel downstream (Figures 2 and 6). Figure 7 illustrates the average grain size distribution of Santa Ana riverbed sediment. Several very important storms occurred in February and March 1980, but none produced floodflows approaching the channel's design capacity. Unfortunately, at the moderate but sustained discharge rates that occurred, and with the potential for sediment starvation, the headward scour of 1978 reappeared in the reach downstream from Seventeenth Street. Figure 2 illustrates that this reach has no grade stabilization structures and an average gradient of 0.0025. During the extended period of discharge, while drawing down the volume of water in storage at Prado Dam, headward scour resulted in a loss of invert sediment for approximately 3 miles downstream from Seventeenth Street. This caused erosion to vertical depths of as much as 18 ft under the Fifth Street bridge in Santa Ana (Figure 8), exposed the bridge's foundation pilings, and required the road to be closed until the stability of the foundation could be reestablished.

The Corps of Engineers, under Public Law 99 (augmented by Orange County Flood Control District funds), awarded a contract to add a series of grouted-rock grade stabilization structures in the damaged area, repair the reinforced-concrete sideslope lining, and backfill the grade stabilization structures using sediment deposited downstream near the ocean in the reach of the channel where the design gradient is approximately 0.0018. The near-ocean deposits reduced the peak flow capacity of the leveed channel; hence the truck haul for upstream backfill served a dual purpose, although it was an expensive solution.

CONCLUSIONS

Several observations appear pertinent to the matter of sediment transport equilibrium.

1. Sediment transport equilibrium requires both a long-term upstream source (deep alluvium) and a downstream disposal mechanism. For the Santa Ana River this would optimally be a sustained sediment-carrying channel velocity, a steeper than natural gradient approaching the ocean, and sufficient littoral drift to avoid in-channel deposition and reduction of the gradient during storms.

2. Bank stabilization, while removing a lateral erosion hazard, also removes a sediment source and can lead to a problem of vertical erosion in downstream locations previously believed to be in a state of equilibrium.

3. The risks of sediment starvation should be recognized when implementing a grade stabilization plan with phased construction, on a "pay as you go" basis by a local agency, over a period of several years.

4. Model studies of grade stabilization and sediment transport equilibrium should be conducted not only for transient peak design discharges but also for intermediate discharges (Figure 4). These lower velocities, which may be sustained for a long period, may not produce erosion over the

sediment source (e.g., the Santa Ana Canyon alluvium) and the consequence can be downstream sediment starvation.

5. Interim flood control measures in a semiarid region may have appealingly low initial costs but provide a false sense of security to lay people unfamiliar with long-term climatic cycles (Figure 3). This is an aspect of risk management that is less than well understood. The general public complains vehemently when low-priced flood control measures fail during statistically predictable occurrences that exceed the level of risk previously adopted.

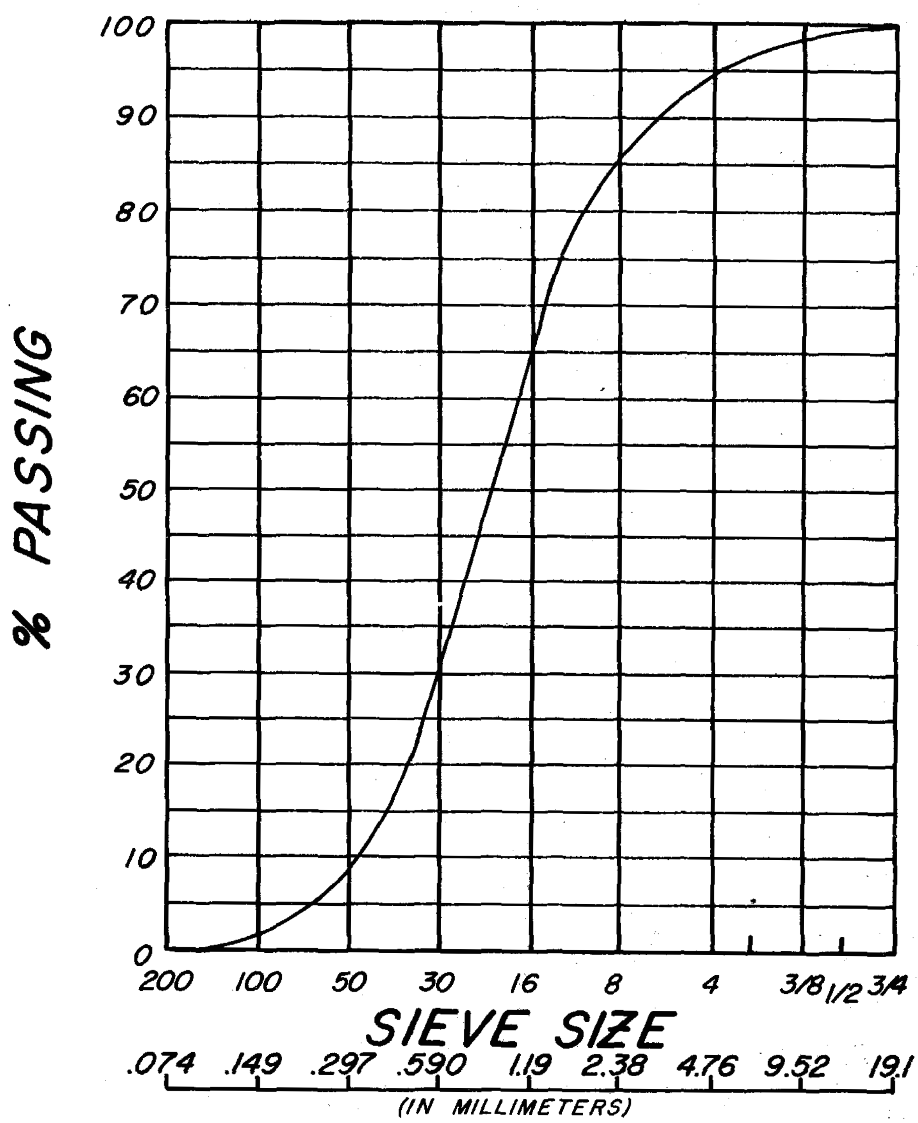


FIGURE 7 Representative gradation of the Santa Ana riverbed downstream of Katella Avenue.

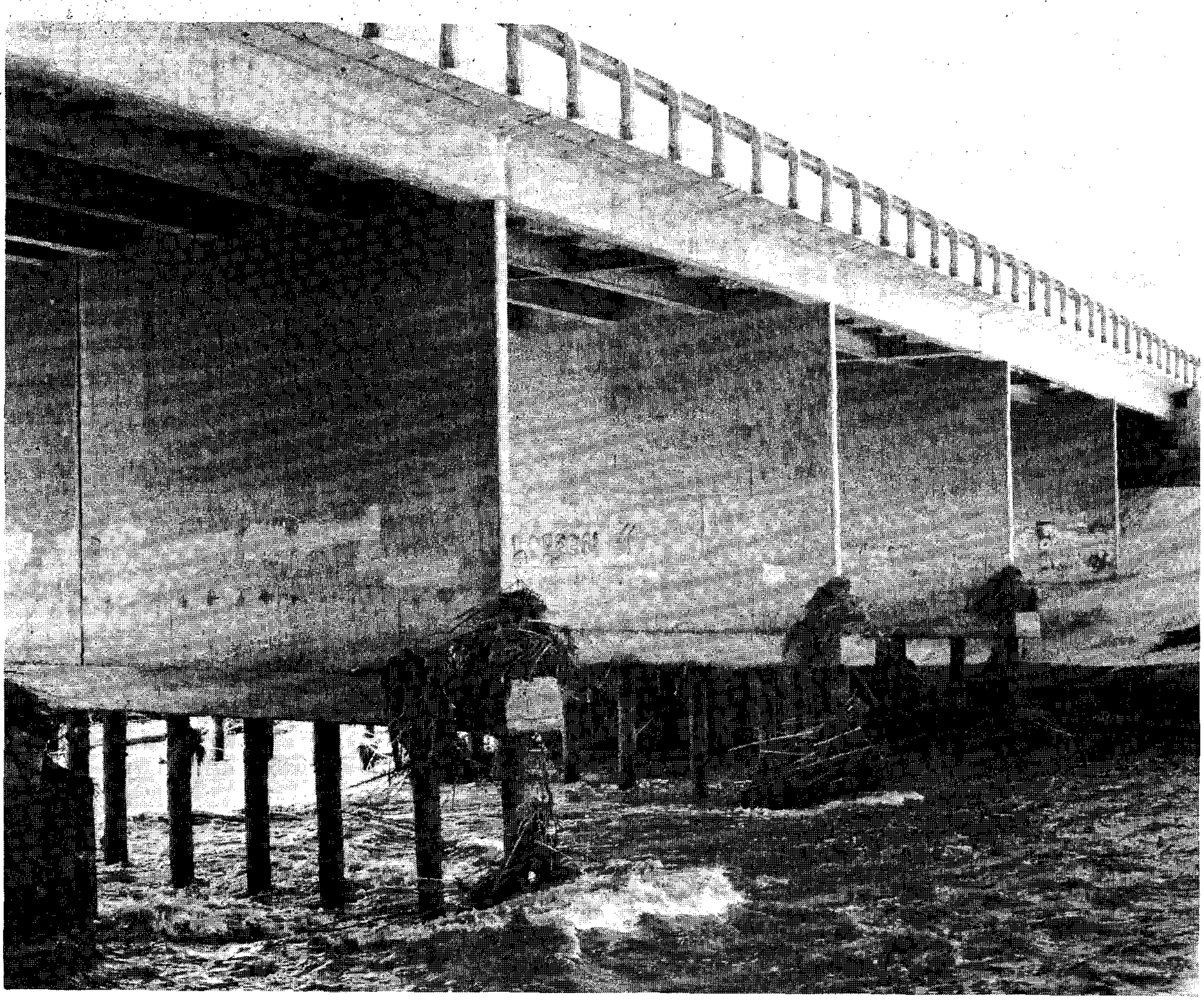


FIGURE 8 Scour of the bed of the Santa Ana River under the Fifth Street bridge in Santa Ana exposed the foundation pilings, as a result of the prolonged discharge of moderate flows from Prado Dam following the floods of February 1980.

The problem on the Santa Ana River is compounded by a deficiency at Prado Dam of storage capacity for the standard project flood. The Corps of Engineers describes this as the largest unresolved flood hazard problem in the western United States. Although the benefit-to-cost ratio is extremely favorable, the approximately $1 billion initial cost of the ultimate solution will be difficult to finance, even at the federal level.

REFERENCES

Orange County Environmental Management Agency, Hydrologic Data Reports, 1977-78 Season (Vol. XIV) and 1979-80 Season (Vol. XVI), Orange County Environmental Management Agency, Santa Ana, California. These reports contain detailed hydrologic data for the 1978 and 1980 flood seasons.

Leeds, Hills, and Jewitt (1964) Flood Control and Water Conservation in Santa Ana River Channel, San Francisco.

U.S. Army Corps of Engineers (1971) Flood Plain Information, Santa Ana River (Imperial Highway to Prado Dam), U.S. Army Corps of Engineers, Los Angeles District, Los Angeles.

U.S. Geological Survey (1980) Water Resources Data for California, Vol. 1: Water Data Report, CA-80-1, U.S. Geological Survey, Menlo Park.

STORM AND FLOOD EVENTS IN SAN DIEGO COUNTY

by Joseph C. Hill and Carey Stevenson

INTRODUCTION

Ronald Robie, Director of the California Department of Water Resources, discusses in his paper "New Approaches to Flood Hazard Mitigation" the progress in developing natural hazard mitigation plans and emphasizes the vital goal of preserving our natural watercourses while at the same time preventing flood damages to developed areas. Through a floodplain management program that has been implemented in San Diego County, more than 200 miles of natural streams have been mapped (see the appendix to this paper). The county's general plan requires that floodways remain in their natural condition unless channelization is necessary to protect structures built before regulations were in force.

Figure 1 shows the rivers that have received floodplain mapping. Symbols have also been added to show the principal areas of flood damage, road damage, and erosion damage during February 1980.

The floodplain maps show excellent correlation to the actual flood areas and provides a valuable basis for rescue and evacuation work in addition to their basic function of regulating construction near flood areas. The accurate location of potential flood areas in relation to the existing street grid has proved extremely useful to police and sheriff units in times of emergency.

FLOODS OF 1980

Information provided in Storm Report, February 1980 (San Diego County Flood Control District, 1980) on rainfall, runoff, and storm damage in San

---

Joseph C. Hill and Carey Stevenson are with the San Diego County Department of Public Works in San Diego, California.

Note: This report is a summary of Joseph Hill's presentation and was prepared by Carey Stevenson. It is based on Storm Report, February 1980 by the San Diego County Flood Control District and includes additional material on floodplain management, storm damage, and flood frequency.

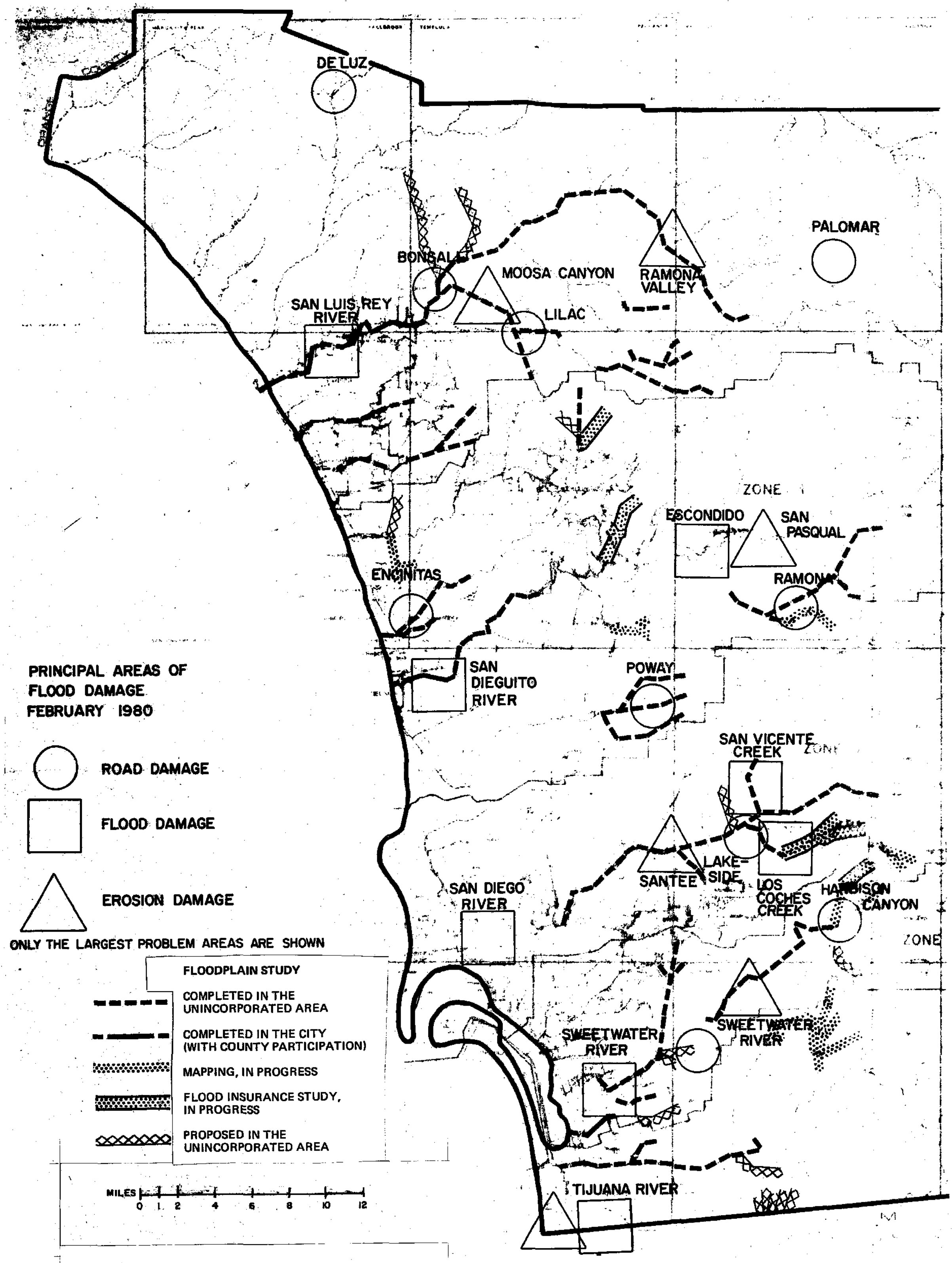
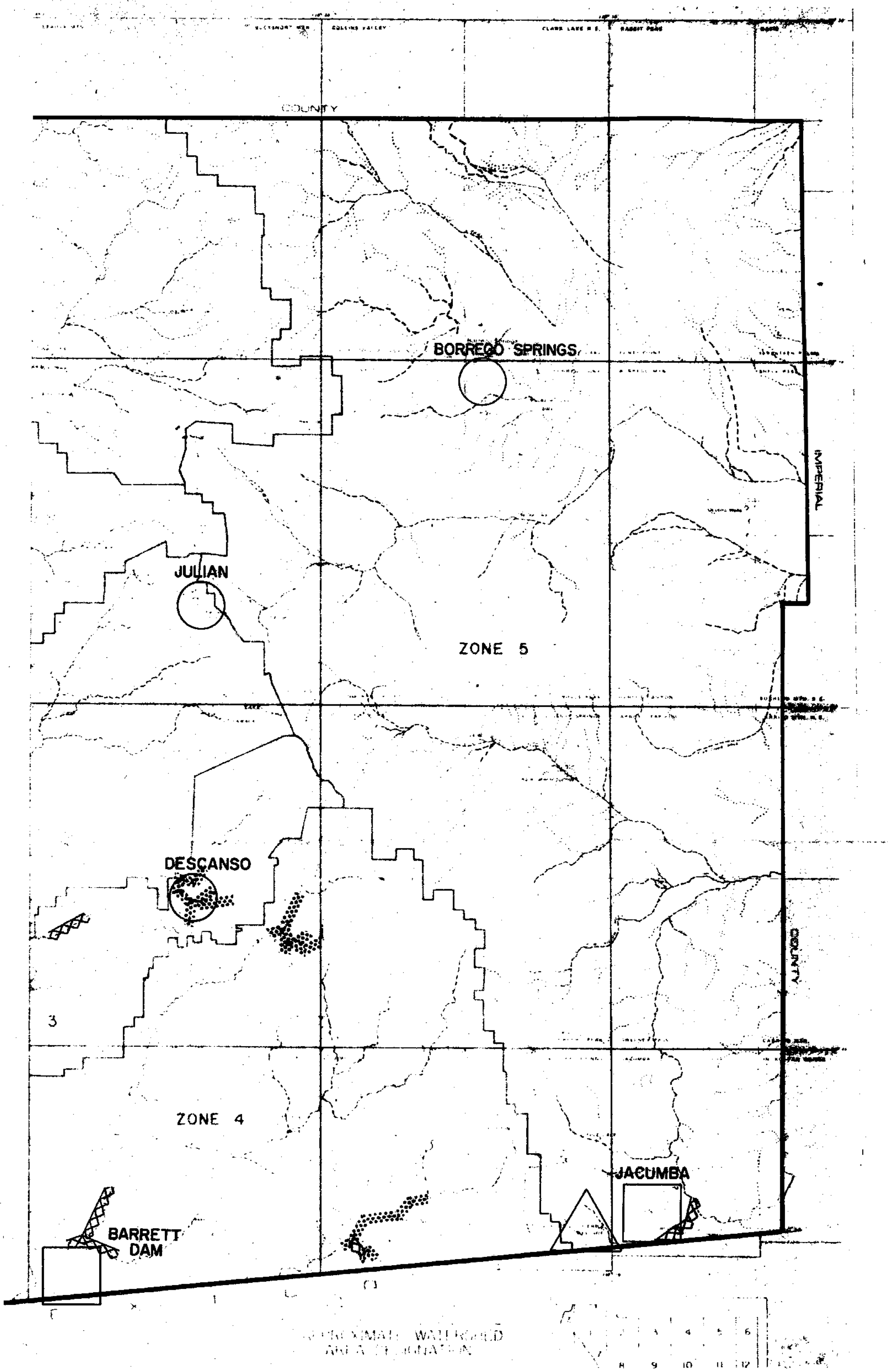


FIGURE 1 Stream size designation map showing the principal areas of flood damage in February 1980.


COUNTY
BORREGO SPRINGS
IMPERIAL
JULIAN
ZONE 5
DESCANSO
COUNTY
3
ZONE 4
JACUMBA
BARRETT
DAM

Diego County was derived from the county's network of rain gages and stream gages plus information from the U.S. Geological Survey and National Weather Service.

The storms resulted from a southward shifting of normal jet stream patterns, as shown by National Oceanic and Atmospheric Administration satellite photographs.

Runoff quantities increased as the rain continued to fall, with the peak flows occurring at the end of the rainy period, when the storm of February 20, 1980, produced large flows in many streams.

Nearly $120 million in damages were estimated in a report prepared for the Federal Emergency Management Agency, and final accounting is expected to show higher totals, particularly in road damages. Disaster declarations were made for San Diego County on February 20, after many reservoirs had begun to spill. Considerable evacuation was necessary, and business in many areas was halted on the afternoon of February 21.

Reservoirs in the county were useful in reducing peak flows in the major streams. For example, San Vicente Dam reduced the peak flow in San Vicente Creek (a tributary to the San Diego River) by nearly half, while El Capitan Dam received more than the 100-year inflow without spilling, excepting minor amounts at a later time. If these flows had passed unimpeded by the dams, damage would have been much more serious. Table 1 shows the approximate frequencies of floods in various locations. The San Diego River-Mission Valley area was one of the principal areas of flooding.

In the last three years nearly all the major rivers in San Diego County have undergone significant changes. Several cases that involve changes in channel bed elevation in excess of 10 ft have been documented (see Figures 2A, 2B, 3A, and 3B).

In some cases channel bed degradation has caused damage at utility crossings and especially at bridge crossings. Piles supporting bridges were partially exposed as a result of channel bed degradation. In one case the Via de Santa Fe bridge on the San Dieguito River was ruptured due to extensive exposure of its support pilings. Examples of erosion are shown in the report Flood Plain Changes During Major Floods (San Diego County Department of Sanitation and Flood Control, 1978).

In other cases sedimentation has affected river channels by filling low areas near crossings, as shown in Figures 4A and 4B.

REFERENCES

San Diego County Department of Sanitation and Flood Control (1978) Floodplain Changes During Major Floods, Department of Public Works, San Diego.

San Diego County Flood Control District (1980) Storm Report, February 1980, Department of Public Works, San Diego.

TABLE 1 Return Periods of Recent Floodflows in San Diego County

Location	Return Period (years)	Peak Floodflow (cu ft/s)	Floodplain Map Completion Date
San Luis Rey River, Oceanside	20 to 30	20,000	Jan. 20, 1976, and Mar. 7, 1977
San Dieguito, Hodges			Nov. 8, 1976
Dam inflow	40	28,000	
Dam outflow	40	22,000	
Ramona	50 to 100		Mar. 22, 1979
Poway	20 to 40	4,500	May 25, 1977
San Diego River			Mar. 5, 1975
San Vicente Dam			
Inflow	25 to 45	11,000	
Outflow	20 to 40	6,000	
El Capitan Dam			
Inflow	50 to 100	40,000+	
Outflow		1,000	
Los Coches Creek (lower)	50 to 100	4,000+	
Forester Creek	10 to 20	4,600	Mar. 15, 1979
San Diego River at Santee gage		9,400	
Spring Valley Creek	10 to 20	--	Aug. 4, 1977
Sweetwater River			1975
Loveland Dam outflow		5,000	
Sweetwater Dam outflow		7,000	

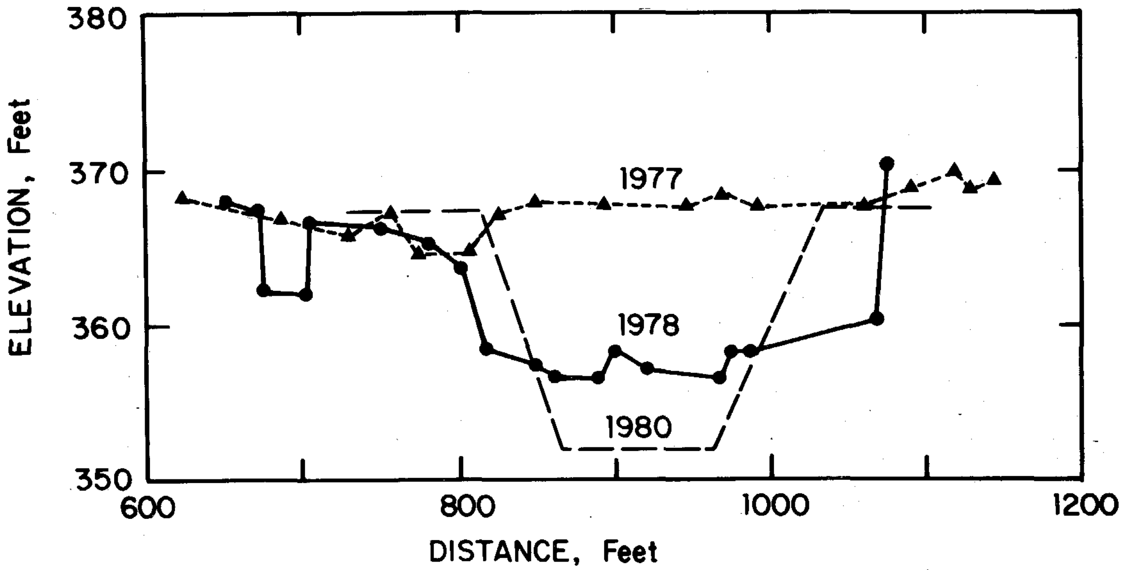


FIGURE 2A Cross section at the damaged location on Riverford Road of the San Diego River.

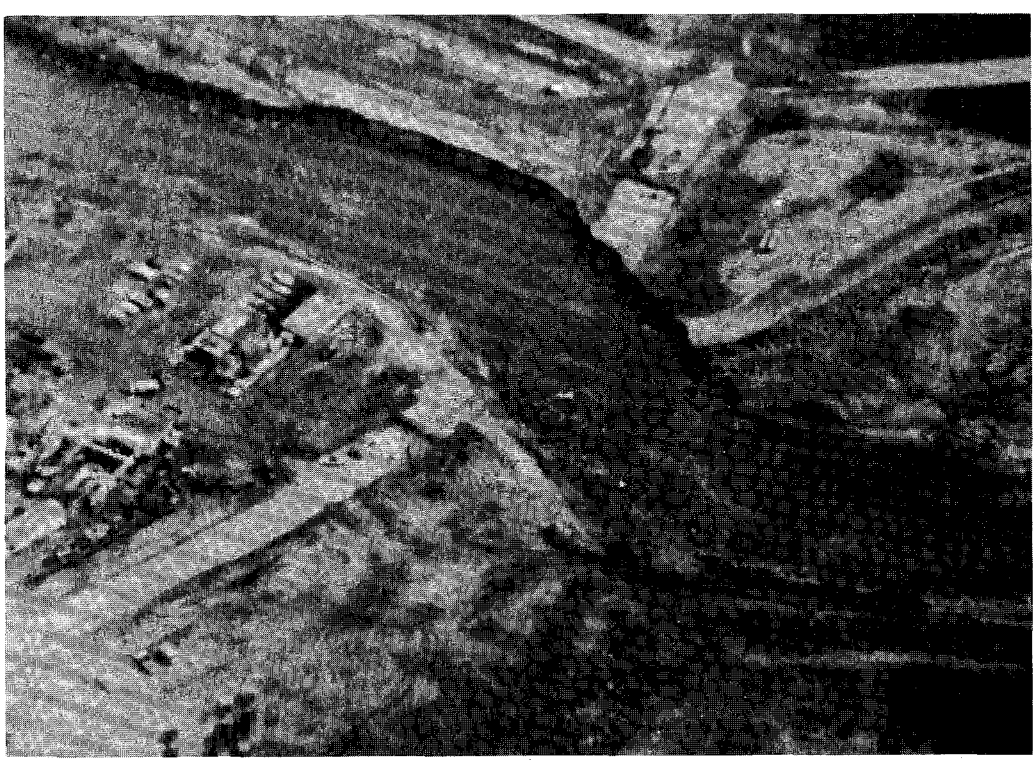


FIGURE 2B Aerial photograph at the damaged location on Riverford Road of the San Diego River.

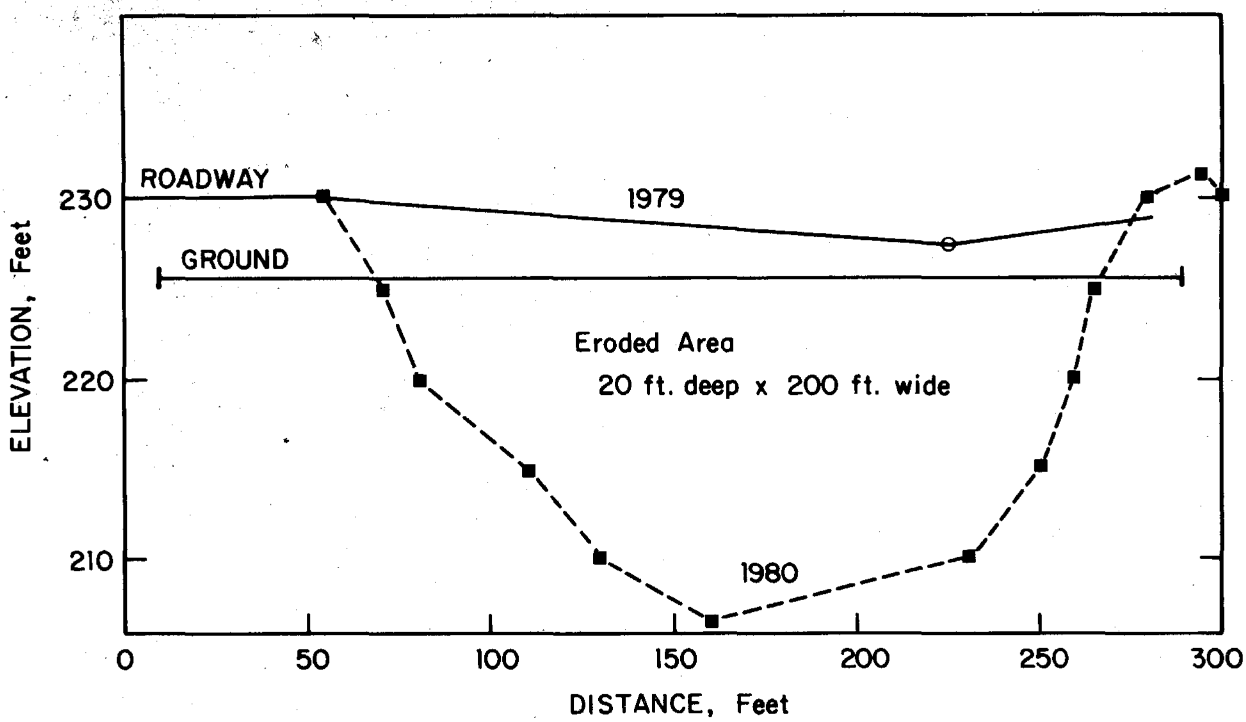


FIGURE 3A Cross section at the damaged location at Camino del Rey of Moosa Canyon Creek.

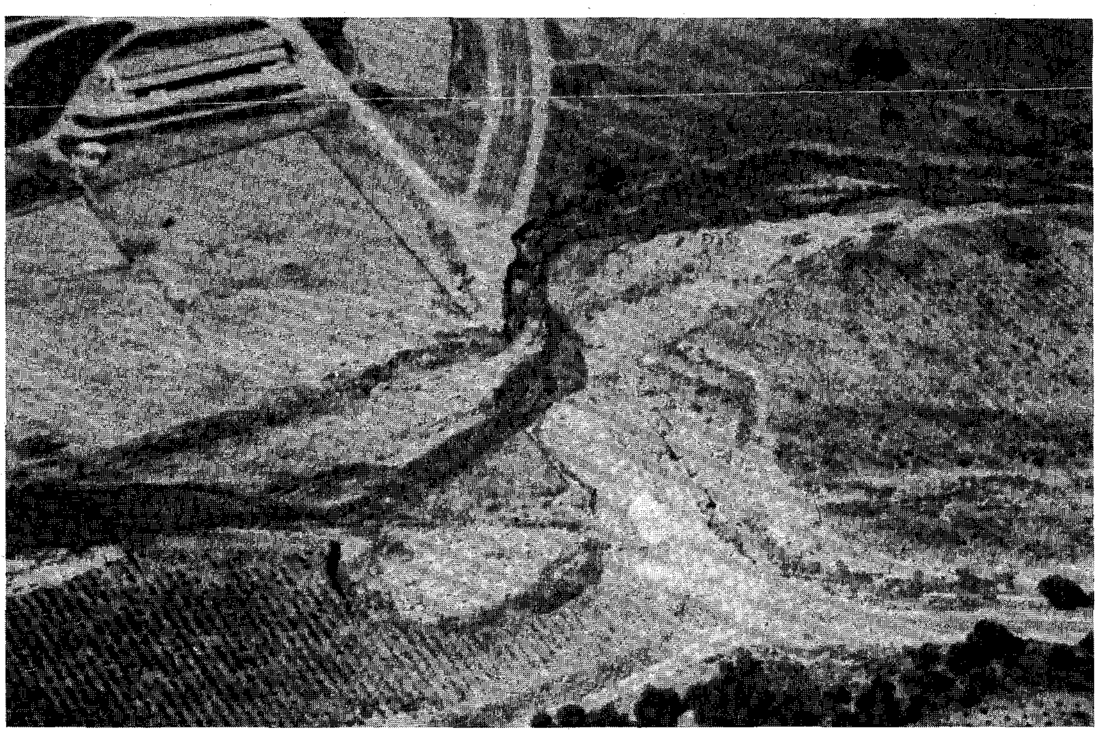


FIGURE 3B Aerial photograph at the damaged location at Camino del Rey of Moosa Canyon Creek.

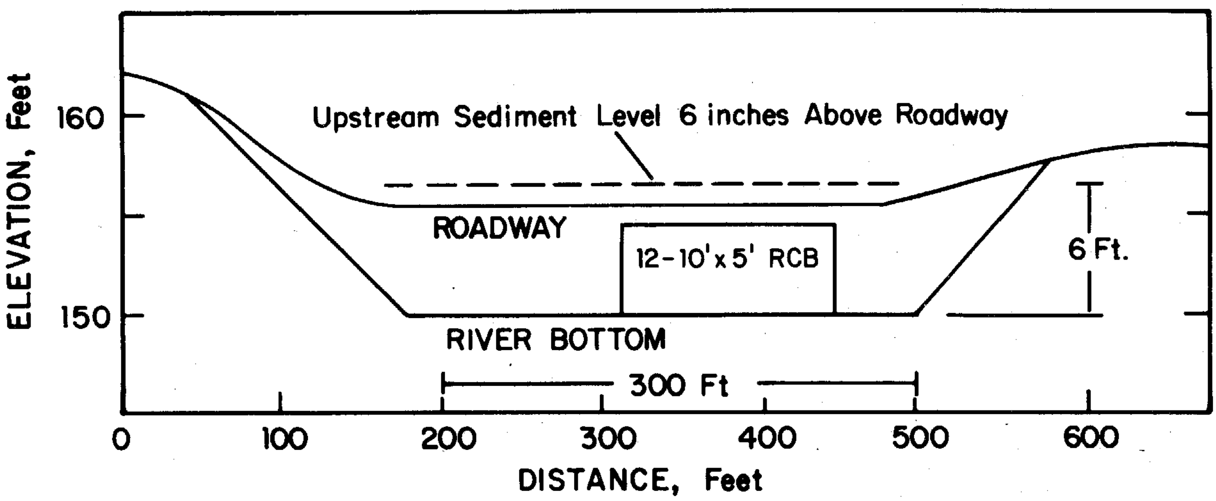


FIGURE 4A Cross section at the damaged location on Olive Hill Road of the San Luis Rey River.

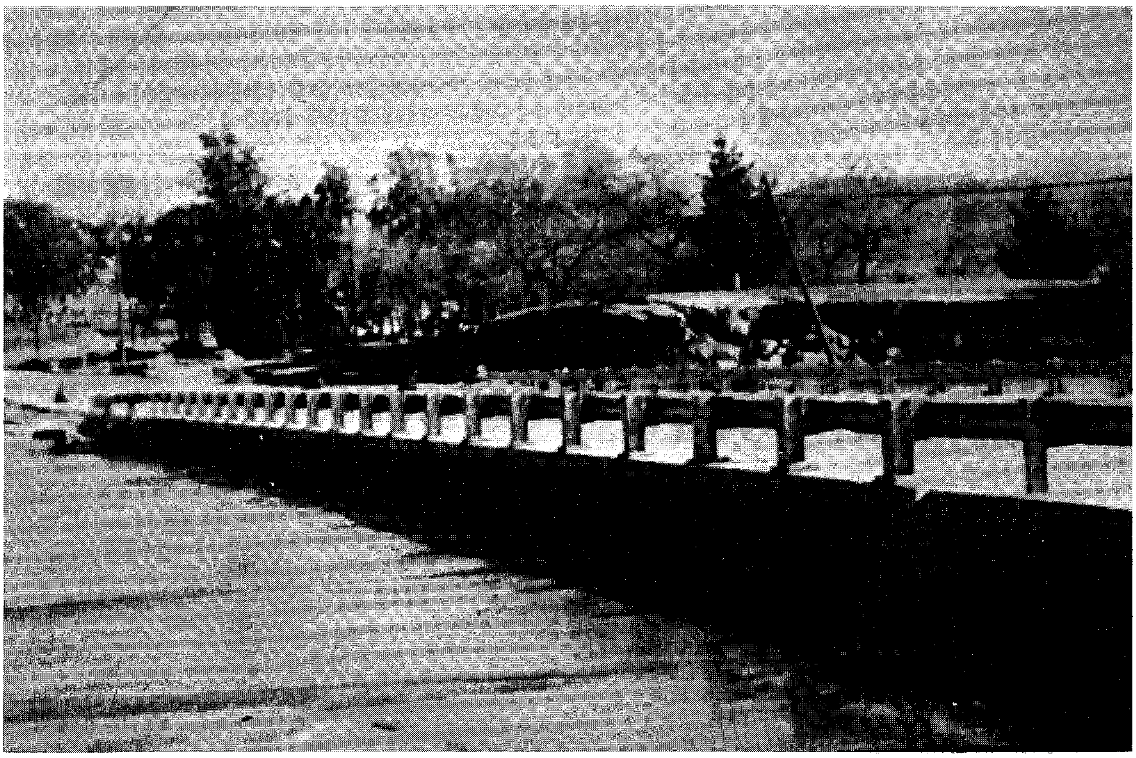


FIGURE 4B Photograph (before sedimentation) at Olive Hill Road of the San Luis Rey River.

APPENDIX: FLOODPLAIN MANAGEMENT USING FLOODPLAIN AND FLOODWAY STUDIES*

HOW IT'S DONE

1. The 100-year floodplain and the floodway are located on a detailed floodplain map like the one shown in Figure A1.
2. City or county government regulates construction in the floodplain by reviewing land development proposals. The floodway cannot be restricted, and the houses constructed in the fringe areas must be above the 100-year flood level.
3. Flood insurance can be purchased for existing buildings in the floodplain through the National Flood Insurance Program.

COUNTY FLOODPLAIN MAPPING PROGRAM

The Board of Supervisors initiated a floodplain mapping program in 1970. The purposes of the program are:

1. Define floodplains and floodways on major rivers and streams.
2. Provide a basis for regulations of floodplains.
3. Provide a basis for planning and zoning.
4. Provide a basis for environmental analysis.
5. Avoid unwise construction in floodplains that would require future construction of channels.

Many of the studies have been extended through incorporated cities as approved by the Board.

RIVERS AND STREAMS WITH STUDIES

Major portions of the floodplains zoned are:

	Miles
Otay River**	12
Escondido Creek	17
Sweetwater River**	18
San Luis Rey River**	37
San Dieguito River**	12
Upper San Diego River**	18
Lower Moosa Canyon Creek	10

---

*This appendix is reprinted from a pamphlet prepared for public distribution by the San Diego County Flood Control District.

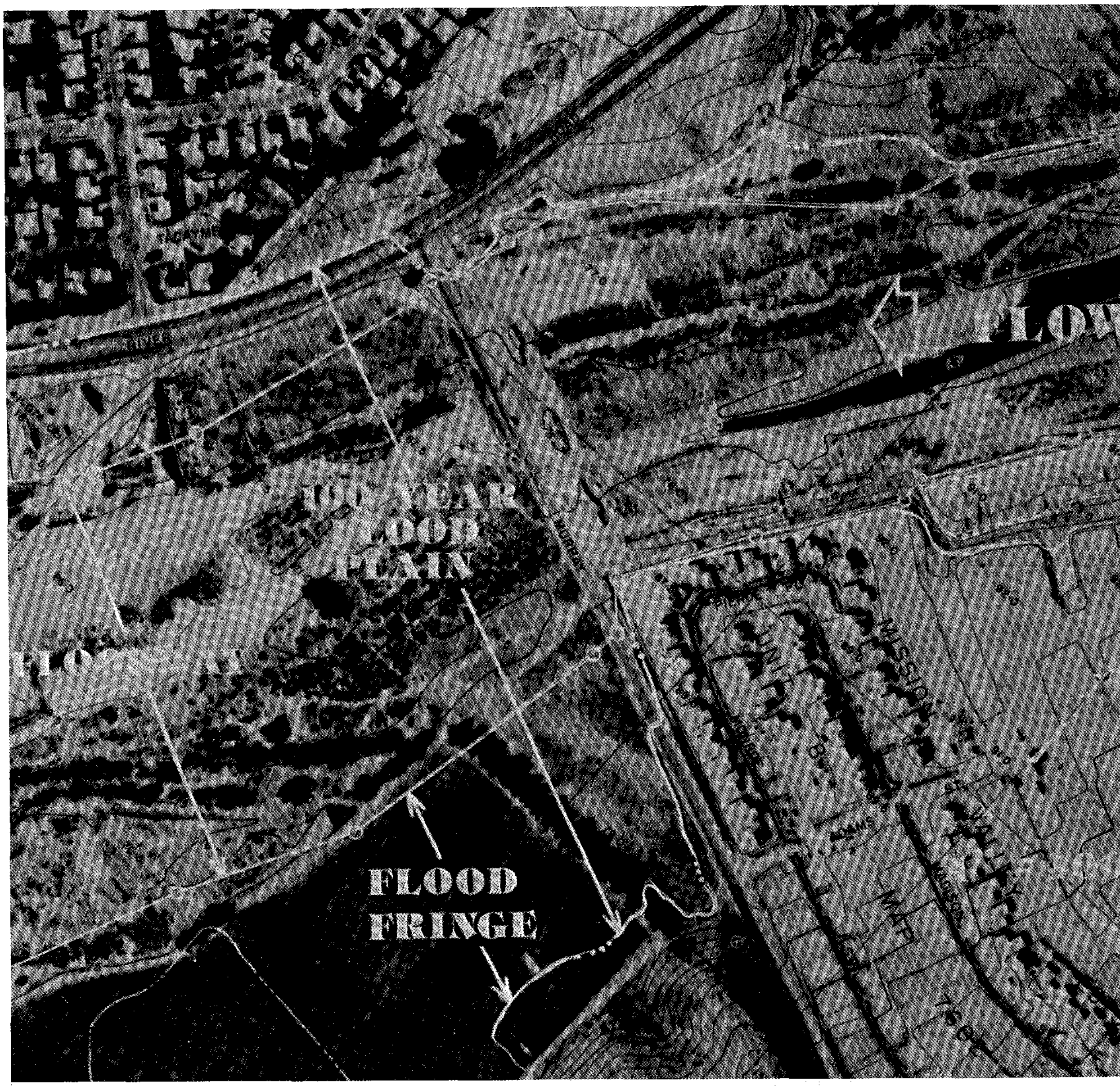


FIGURE A1 Typical floodplain map.

Other streams with studies (some in progress) are:

Poway Creek	15
Riedy Creek**	4
Keys Canyon	9
Buena Vista**	5

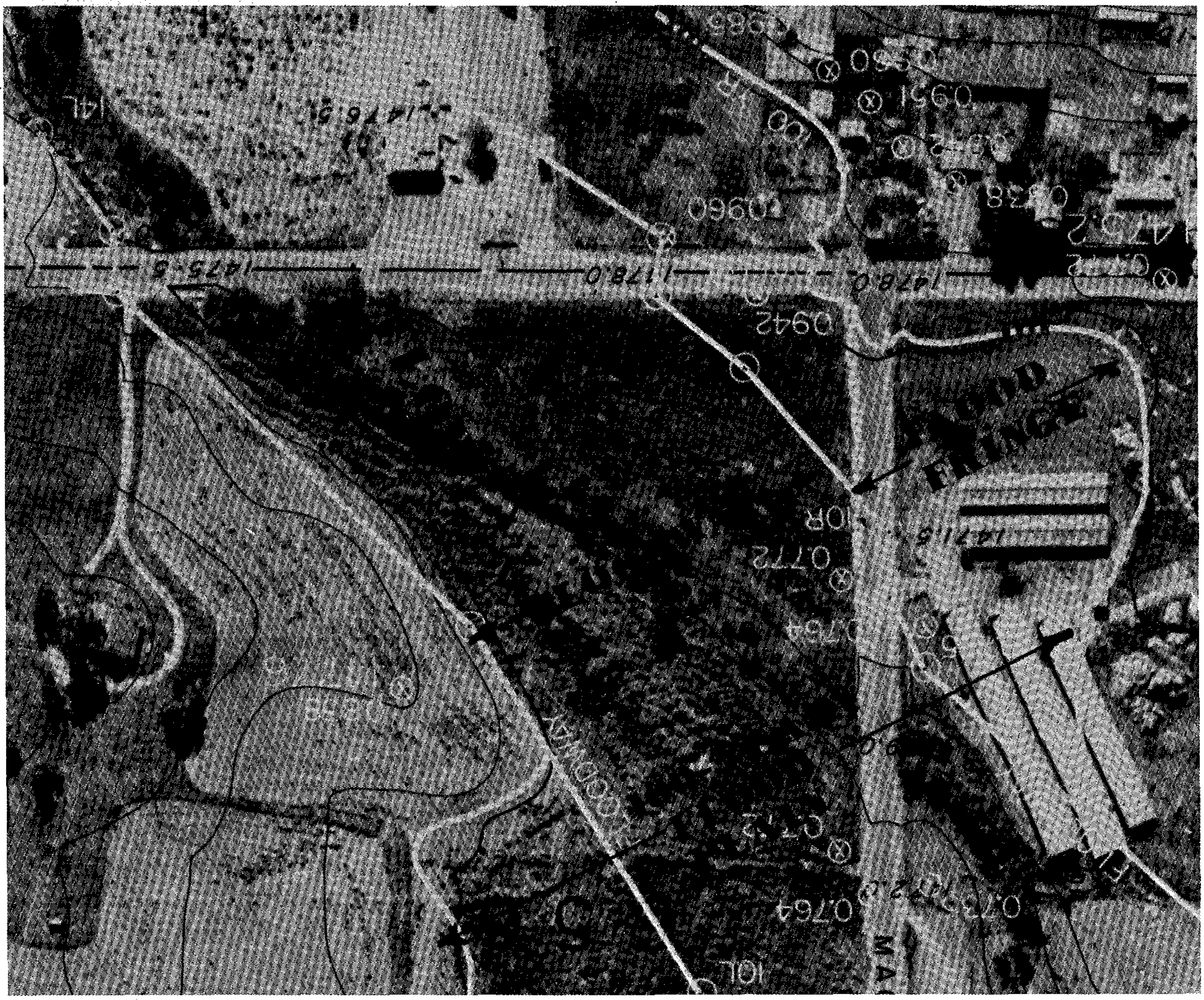


FIGURE A2 Flood map (inverted to match the photograph in Figure A4). Note the relation of the flood lines to the store at the top of the map and to the poultry ranch at the right.

Alvarado Creek**	4
Agua Hedionda**	8
Telegraph Canyon**	3
Poggi Canyon**	3
Spring Valley Creek	6
Upper Moosa Canyon	5
Santa Maria Creek	10
Middle San Diego River**	4

**Includes areas in cities.

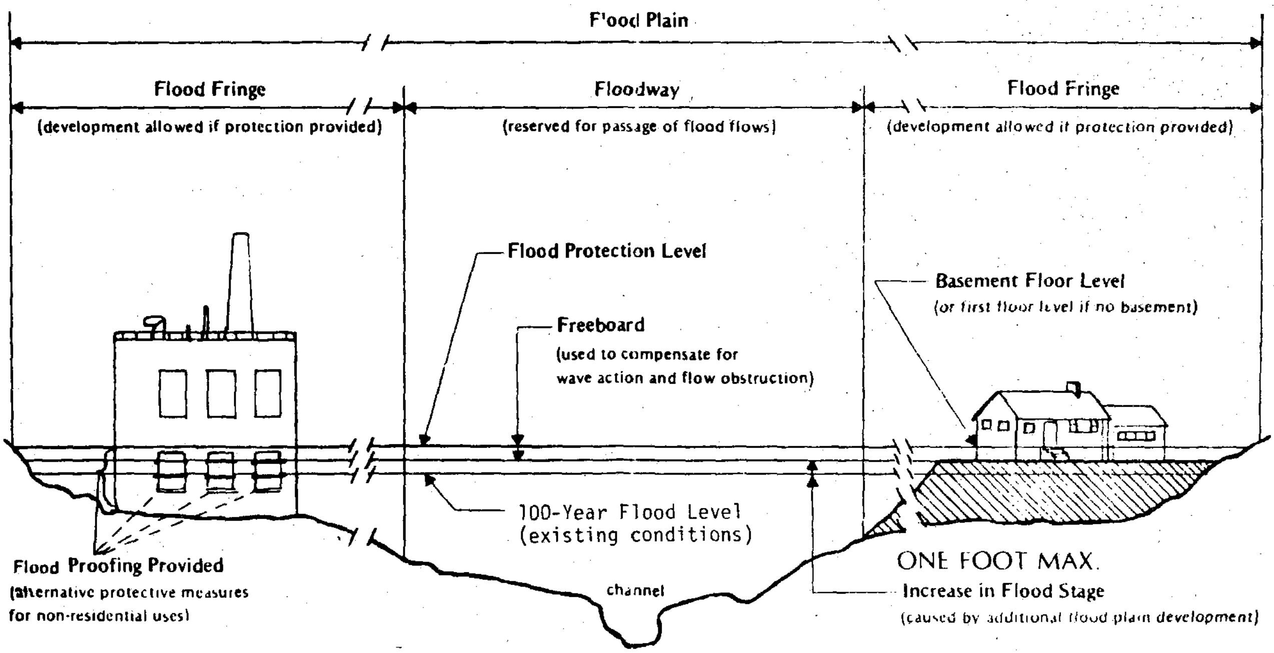


FIGURE A3 Valley cross section.

"100-year flood" means a flood estimated to occur on an average of once in 100 years (one percent probability of occurrence each year).

"Floodplain" means a land area in and adjoining a river, stream, watercourse, ocean, bay, or lake that is likely to be flooded.

"Flood fringe" means all that land lying with the 100-year floodplain that is not within the floodway.

"Floodway" means the river channel and the adjacent land areas required to carry the 100-year flood, without increasing the water surface elevation of that flood more than 1 ft at any point. Additional criteria used in the design of the floodway may be specified by the local government.

NATIONAL FLOOD INSURANCE PROGRAM

The National Flood Insurance Program (NFIP) was started in 1968 by an act of Congress, with major amendments in 1973 and 1977. The program is administered by the Federal Insurance Administration (FIA). Citizens can buy flood insurance at subsidized rates if their community is participating in the NFIP.

Detailed studies prepared for the FIA include maps with insurance zones. After these maps are issued, insurance rates vary and are as low as $0.01 per $100 for structures outside designated flood areas. Within designated flood

FIGURE A4 Aerial photograph taken by the Ramona Sentinel in February 1980 after a flood of nearly 100-year frequency. Magnolia Avenue is in the foreground, Highway 78 is in the distance.

areas, lending institutions regulated by the federal government must require flood insurance as a condition for issuing loans.

Limits of Coverage and Subsidized Rates (dollars)

Type of Structure	Structure Maximum Coverage	Structure Rate/$100	Contents Maximum Coverage	Contents Rate/$100
Single family	35,000	0.40	10,000	0.50
Other residential	100,000	0.40	10,000	0.50
All nonresidential	100,000	0.50	100,000	1.00

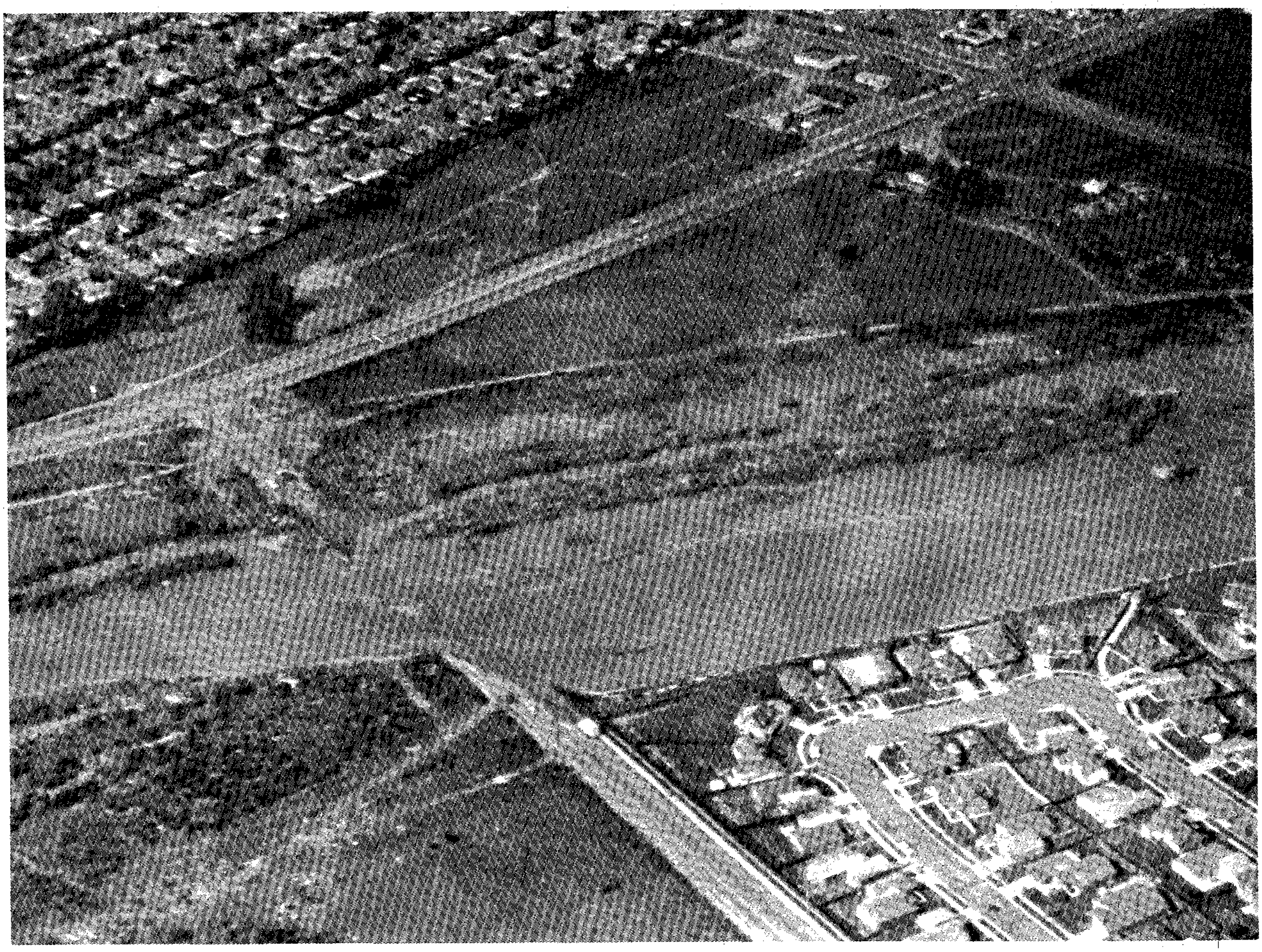


FIGURE A5 Photograph of the 5- to 10-year flood of January 17, 1978, taken in the same area as the flood map shown in Figure A1.

For information call the County of San Diego Department of Public Works, 5555 Overland Avenue, San Diego, California 92123 (714) 565-5120.

COMMENT ON PEAK FLOODFLOWS IN SAN DIEGO'S MISSION VALLEY, 1978-80

by Philip R. Pryde

The San Diego River floods of 1978, 1979, and 1980 impressed the citizens of San Diego in many ways. Among the most evident of these were the obvious effects of urbanization on downstream runoff patterns. So great was the increase in maximum Q in Mission Valley, as compared with that a few miles upstream, that one must ask if these effects are perhaps not underestimated.

In 1976 the Corps of Engineers recalculated flood frequencies on the lower San Diego River and determined the 50-year flood size to be 17,000 cu ft/s. Within a three-year period this figure was closely approached or exceeded four times (Table 1). The Corps considered the effects of expected basin urbanization in deriving their figures, but considering that almost all of this flooding was produced by storms of recurrence intervals much less than the corresponding flood recurrence intervals, it seems reasonable to ask whether the accepted discharge-frequency curves showing the effects of basin urbanization may not in fact understate reality. In 1979, for example, 15- to 25-year 24-hour precipitation in La Mesa and East San Diego produced a 50-year flood in Mission Valley.

Viewed another way, the earlier large floods on the San Diego River (e.g., 1916) were estimated to be less than 10 percent larger in Mission Valley than at the official gaging station in Mission Gorge (10 miles upstream). However, the 1978 flows were as much as 500 percent larger and surprised almost everyone by their magnitude. Furthermore, at no time did spillage from upstream dams account for more than a small fraction of the Mission Valley flow (El Capitan Dam, the largest, was not overflowing at the time of any of these peak flows). Additionally, the lower basin is still a long way from being at a stage of maximum development.

In light of the above it would seem appropriate to inquire if another look at the existing assumptions about the effects of urbanization on downstream flood runoff is not in order. The San Diego experience of the last three years would seem to suggest that these effects might be greater than generally assumed.

Philip R. Pryde is Professor in the Department of Geography at San Diego State University in San Diego, California.

TABLE 1 Storm and Runoff Data for Mission Valley Floods of 1978-80

	Peak Discharge (cu ft/s)		Maximum Precipitation at SDSU[a] (in.)		
Flood Date	Mission Valley	Mission Gorge	24-hour	48-hour	Previous Week
Jan. 15, 1978	15,000	3,010	2.34	2.94	4.38
Mar. 1, 1978	14,000	2,480	1.21	1.81	2.82
Jan. 31, 1979	17,000	Gage not working	3.30[b]	3.56	3.65
Feb. 21, 1980	27,000	Gage not working	1.65	2.16	4.82

[a] The San Diego State University (SDSU) gage is 6 miles upbasin from the site of the flow figures given for Mission Valley. The SDSU 100-year 24-hour storm equals about 5 in.

[b] The 24-hour precipitation was 4.25 in. in La Mesa (3 miles upstream), which equals a 25-year storm.

1980 FLOODS IN THE SACRAMENTO-SAN JOAQUIN DELTA

by Charles A. McCullough

Levees failed within the same hour at two large delta islands in the Sacramento-San Joaquin Delta in January 1980, flooding about 10,000 acres. This occurred shortly after the highest tide in a two-week period. Superimposed on the tide stage was an additional 2 ft caused by an inflow to the delta of about 300,000 cu ft/s, and this was accompanied by north winds of about 60 miles an hour.

This paper describes the effort to save interior levees of the flooded tracts from erosion by waves generated by winds blowing across the open bodies of water, a second period of high inflow in February and additional flooding that accompanied it, the emergency levee repairs on numerous delta islands, the Corps of Engineers' Public Law 99 decision for the delta, the declaration of emergency by the Federal Emergency Management Agency (FEMA) and some of the differences between this and a declaration of a natural disaster, the actions to close the breaks in the levee and to pump the water out of the flooded tracts, and the hazard mitigation program proposed to decrease the probability of recurrence of flooding of the two tracts.

The 1980 flood in the Sacramento-San Joaquin Delta occurred in about an hour's time in the late afternoon of January 18. Heavy rains had started January 10 and by January 18 had built up inflow to the delta from its usual wintertime figure of 550 cu m/s (20,000 cu ft/s) to about 8,500 cu m/s (300,000 cu ft/s).

This flow raised the level of the water surface in the delta about 0.6 m (2 ft) above the biweekly high tide level, which also occurred on January 18. This high water level was accompanied by north winds of nearly 100 km (60 miles) per hour. The Webb Tract levee failed first, opening a breach about 250 m (800 ft) wide in the levee and flooding about 2,200 ha (5,400 acres). The north levee of Holland Tract failed about an hour later, opening a 75-m (250-ft) breach and flooding more than 1,600 ha (4,000 acres). The few residents on Webb Tract were in buildings on the edges of the levee and were

---

Charles A. McCullough is Chief of the Division of Flood Management for the California Department of Water Resources, Sacramento, California.

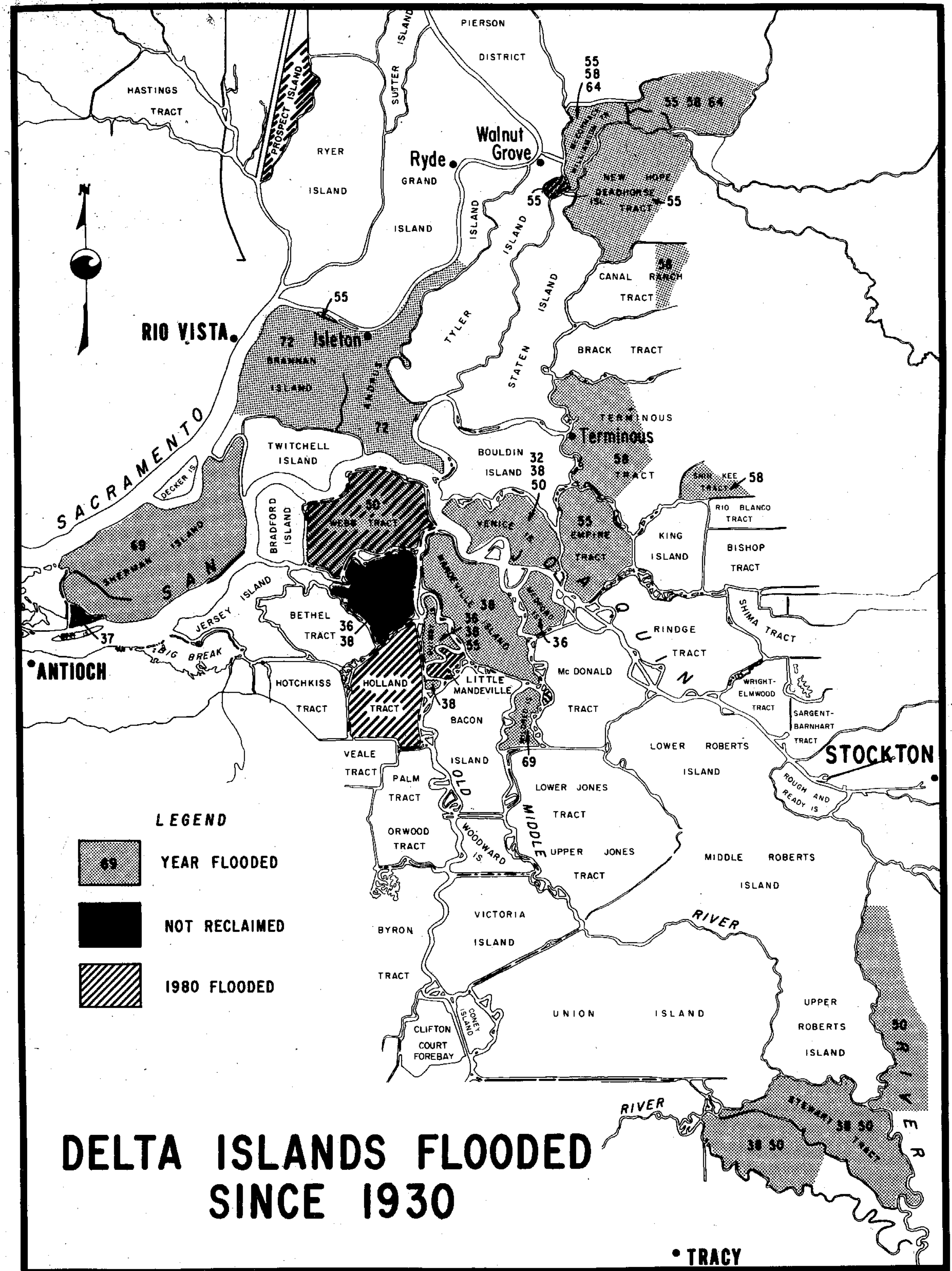
not injured. There were some residents on the lower-lying land in Holland Tract, and they had to flee the rising water. There were a few injuries and one duck hunter has not been found. In addition, more than half of the 1,500 head of beef cattle on Holland Tract drowned.

The levees around the land in the delta (the land is sometimes called tracts and sometimes islands) were built around 55 tracts about 100 years ago. At that time the land elevation was at about mean tide level--underwater during the highest tides and out of water during low tides. In the ensuing 100 years the levees have changed their character to flood control levees as the land on the islands or tracts subsided due to oxidation and compaction of peat or its removal from the area by wind erosion. The centers of the islands are now as much as 6 m (20 ft) below sea level. Although the levees constantly subside as the peat beneath them compacts or flows out from under the levee section, the failures are almost always structural rather than the result of overtopping. The failure at Webb Tract was clearly structural. At Holland Tract there seems to have been a combination of wave erosion and structural failure. Figure 1 shows the location of the Holland and Webb tracts and also the history of flooding of delta islands since 1930.

As soon as the tracts flooded, a new problem arose. The broad expanse of water inside the flooded tracts, along with the strong north wind, resulted in high waves at the south end of the tracts (Figure 2). This was particularly damaging to the interior of the south levee at Holland Tract, and Reclamation District 2025 immediately arranged for rock to be hauled in to protect the levee and the road on the levee. It was essential that these levees be preserved in order that it be physically possible to reclaim the tracts. The California Department of Water Resources implemented its agreements with the California Department of Forestry and the California Conservation Corps to put flood fighters on the levees, who installed canvas and sandbags at critical spots to protect levees from wave damage. The Corps of Engineers started its flood fight process. It is still continuing to supply technical assistance, equipment, and materials to rehabilitate the threatened levees in an attempt to avoid further damage and flooding. Its efforts have been funded by the Federal Emergency Management Agency (FEMA).

Attempts were also made to have the President declare a "major disaster" for the area. This effort resulted on February 1 in a "declaration of emergency," which authorized FEMA to do some of the things it would have been able to do under a major disaster declaration. Costs incurred by state and local agencies prior to February 1 are not eligible for federal reimbursement. The cost of pumping out the islands is eligible under this declaration only to the extent necessary to prevent damage to the levees from wave action across the flooded tracts. The additional cost of pumping to drain the islands below the water level at which this damage ended has to be borne by the local reclamation districts. The federal rationale for this more limited declaration seems to have some of the elements found in Lewis Carroll's story Alice in Wonderland. Certainly, no one who has worked with FEMA on this flood event has been able to obtain from the agency an understandable explanation of the basis for this decision.

A second Alice in Wonderland-type federal decision was reached when



Department of Water Resources 1980

FIGURE 1 Sacramento-San Joaquin Delta islands flooded since 1930.

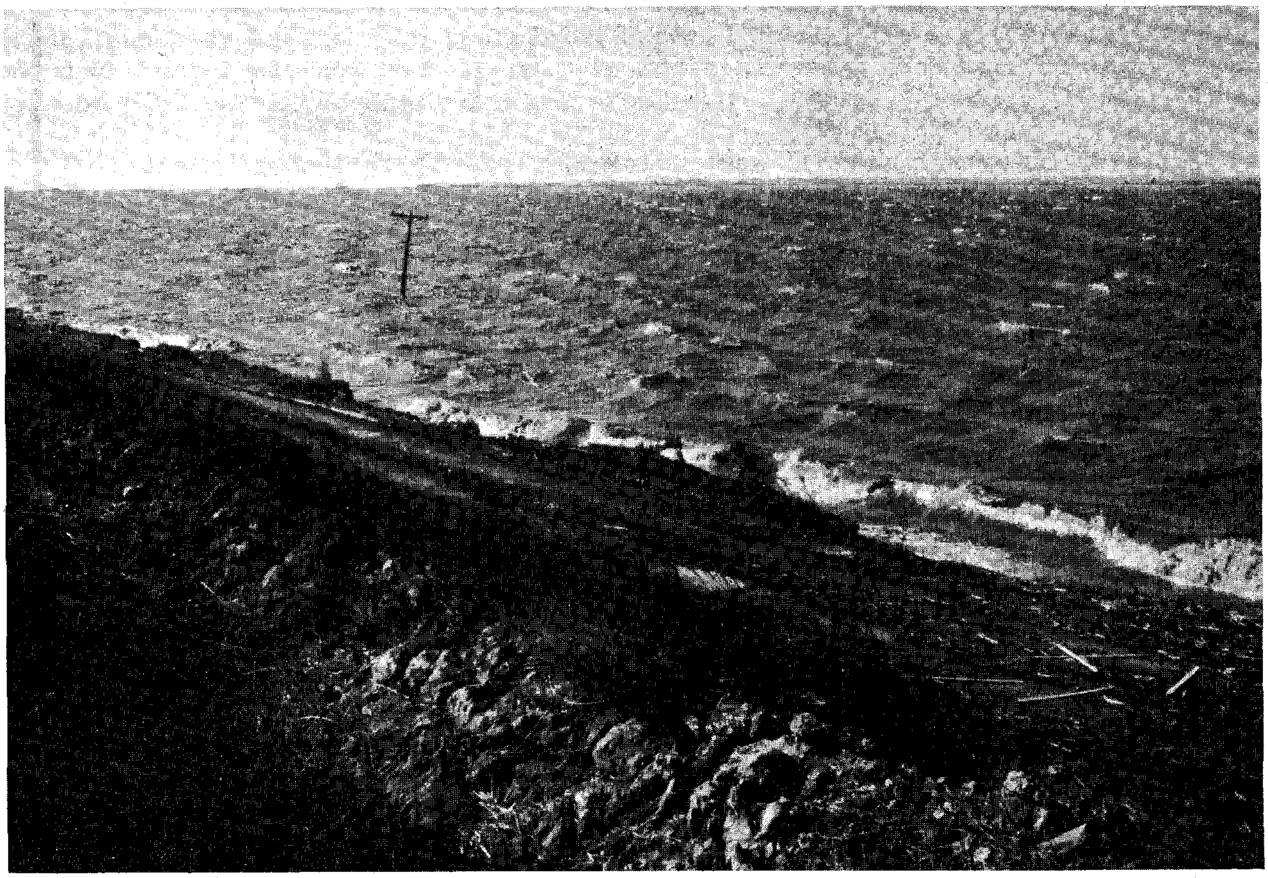


FIGURE 2 Wave damage to interior levee of a flooded tract.

General Morris, Chief of the Corps of Engineers, concluded that the delta is not eligible for emergency flood assistance under Public Law 99. The General explained his rationale during a tour of the flooded area by saying it was based on the fact that the levees were originally built to reclaim the land. This interesting reasoning seems to ignore 100 years of change, in which the lowering of the land level of the tracts and islands has created a situation in which they are exposed to the threat of flooding; it also ignores the crucial impact of a 8,500-cu m/s (300,000-cu ft/s) flood inflow to the delta, which superimposed 0.6 m (2 ft) of additional water level on the recurrent bimonthly highest tide levels.

The immediate question for all the public agencies concerned was whether the expenditure to close the breaks and pump the water out of the islands would be justified. The first benefit of course would be the value of the agricultural land in farmable condition as contrasted with its meager value under a lake. Maintenance of satisfactory water quality in the southern portion of the delta was also threatened because of the flooded tracts. If left as permanent lakes, the surrounding levees would soon be eroded away. This would provide a much shortened path for ocean salinity to intrude to the State Water Project pumping plant and the two Central Valley Project plants.

This was especially serious at Holland Tract, which is nearest the plants. Reclaiming the tracts avoids the water quality problem and is a second major benefit.

During the summer in all years, and almost year-round in drought years, the fresh water consumed in the delta is released from upstream storage reservoirs. About 6,000 cu m/ha (2 acre-ft/acre) more water evaporates each year from an open water surface in the delta than from evapotranspiration from land and crops. The additional water would have a very high value, and preventing this loss was a major benefit to justify closing the breaks and pumping the water from the tracts. On the basis of these benefits (see Table 1 for a summary of costs and benefits), FEMA authorized the Corps of Engineers to design and implement a program to drain the two tracts.

TABLE 1 Costs and Benefits from Reclamation of Webb and Holland Tracts (dollars)

	Estimated Cost	Actual Cost	Estimated Annual Cost	Estimate of Annual Benefit
Webb Tract	10,000,000	7,330,000[a]	1,500,000	2,850,000
Holland Tract	3,700,000	3,238,000	620,000	5,720,000

[a]The dewatering cost is an estimate.

On February 13 a second series of intense storms began in the same area of the state, resulting in another 8,500-cu m/s (300,000-cu ft/s) inflow to the delta. These storms were accompanied by very strong westerly winds and low barometric pressure. The delta water levels were increased by the wind and low-pressure effects as well as by the 0.6 m (2 ft) of additional stage caused by the inflow.

The threat to many of the delta islands mounted rapidly, and in short order all agencies were fully involved in flood fights on the many tracts and islands threatened by the high water and wave erosion. Nearly all of the floating construction equipment in the delta--dredges, barges, barge-mounted drag lines, and tugs--was at work, along with many land-based construction firms.

Approximately 200 California Conservation Corps personnel and a smaller number of California Youth Authority Conservation Camp personnel carried out sandbagging and wave wash protection work on numerous islands under Department of Water Resources supervision (Figure 3). A camp for the California Conservation Corps personnel was established at Antioch by the California Department of Forestry. A Department of Corrections crew manned the kitchen.

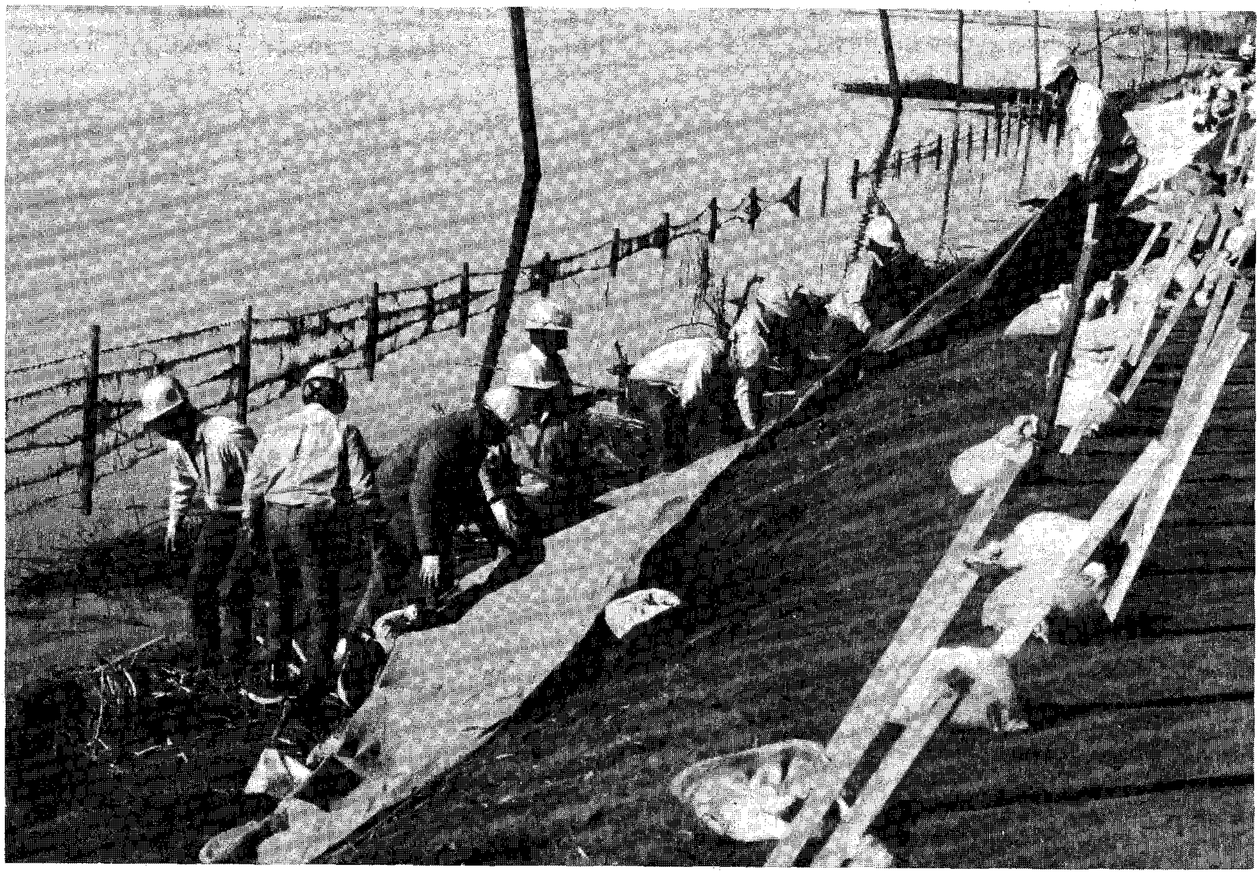


FIGURE 3 Wave wash protection by the California Conservation Corps.

The California National Guard set up a portable shower at the camp; the National Guard also supplied helicopters with crews for numerous inspection and coordination flights in the delta. The California Office of Emergency Services coordinated the work of a number of the agencies. An equally large group of federal and county agencies participated in various phases of the work.

The magnitude of the effort is indicated by the claims for emergency repairs approved by FEMA for some 33 islands and tracts exclusive of Webb and Holland, a total of about $5.25 million.

These efforts were largely successful. Only two small tracts suffered levee failures--Dead Horse Island, with about 80 ha (200 acres), and Prospect Island, with about 450 ha (1,100 acres).

Several news articles in the months following the flood fight mentioned financial problems of private contractors and materials vendors who had not been paid by the local reclamation districts. The districts reported they had authorized the expenditures on verbal commitments of FEMA personnel after expending all district funds. The problem seems to be one of timing. There

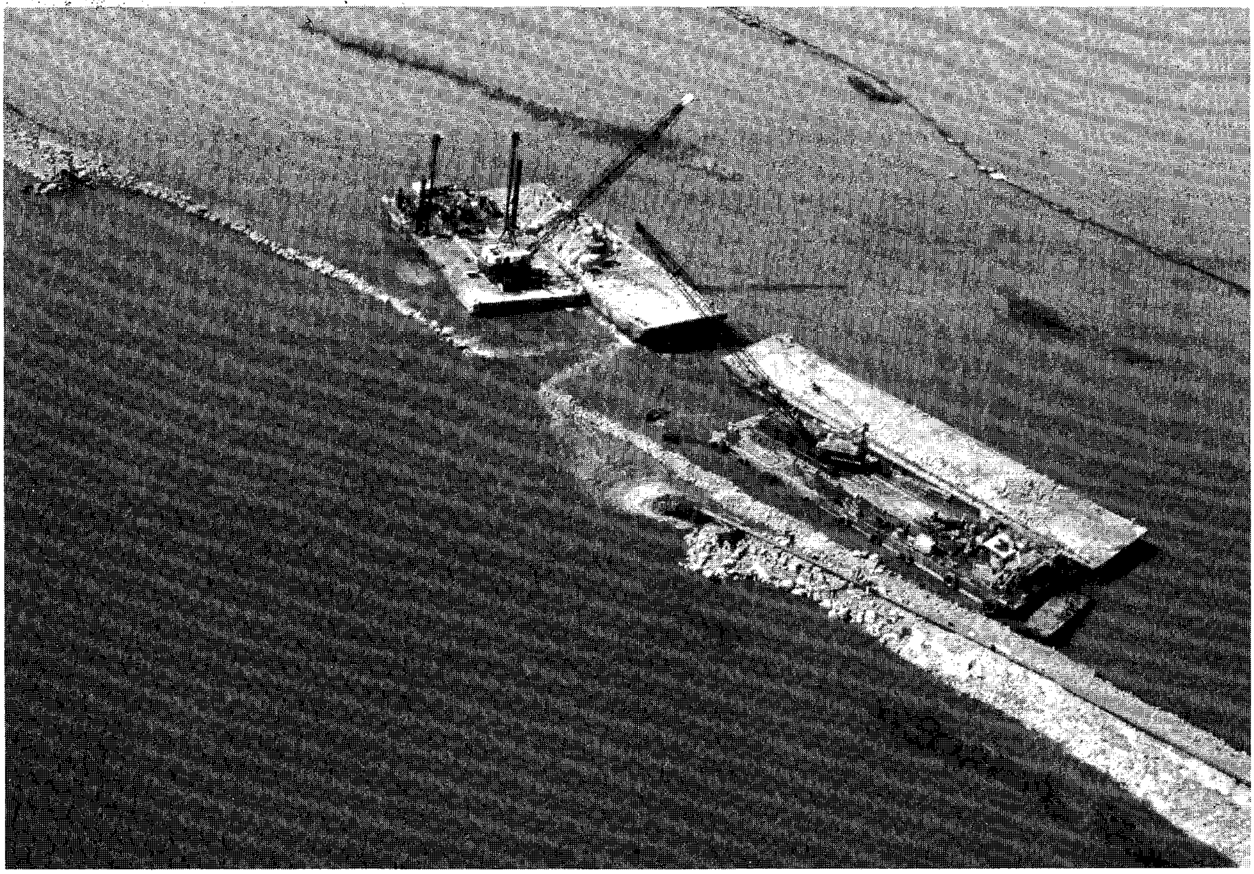


FIGURE 4 Sand and rock closure of Holland Tract.

was no suggestion in the articles that the commitments would not eventually be honored.

The two small inundation areas have been reclaimed, one by the landowner and one with reimbursement from FEMA. Holland Tract was reclaimed by the Corps of Engineers under work assignments from FEMA and by Reclamation District 2025 in time for some crops to be planted in the late summer (Figure 4). The closure of the break at Webb Tract, by the Corps of Engineers under work assignment from FEMA, is completed and pumping was expected to start in early October 1980.

In accordance with a FEMA requirement for hazard mitigation, Reclamation Districts 2025 for Holland Tract and 2026 for Webb Tract plan to improve their levees to a significant degree to decrease the possibility of such failure in the immediate future. These plans, developed with the Department of Water Resources and the Corps of Engineers, now await action by the Office of Emergency Services, FEMA, and the reclamation districts.

The Department of Water Resources and the Corps of Engineers have had a detailed flood management study for the delta under way for several years.

This study is expected to be completed in 1982. It will show the areas where better flood protection in the delta can be justified on an economic and social basis.

There are three matters I alluded to earlier that I suggest be given further consideration. The first is the criteria for a declaration of emergency as contrasted with declaration of a major disaster by the federal government. The rationale for this decision certainly needs to be made available in an understandable manner for public consideration. The second is the interpretation of Public Law 99 by the Corps of Engineers as applied to the Sacramento-San Joaquin Delta. The rationale for the Chief of Engineers' decision needs to be reviewed in the public forum. Finally, the procedure for partial payments to local districts needs to be examined to ensure that the payments are made as promptly as the law permits.

The positive note from this flood disaster is the policy being enforced by FEMA of implementing actions to mitigate the disaster situation so that the next flood in this region will be less damaging than the last one. If the actions live up to the promise, this may rank with the flood insurance program in value to the state and the nation.

SEDIMENT FLOW HAZARDS: SPECIAL HYDROLOGIC EVENTS

by John M. Tettemer

This paper describes the problem of sediment-laden floodflows as a major component of flood damage in the arid areas of the Southwest. The problem is defined in terms of its nature, the factors involved, the areas affected, and the impacts on individuals, communities, and government.

Engineering aspects are discussed, including factors involving sediment production, prediction models, and hydraulics. Control measures include maintaining the capability for sediment to move through the system or installing traps to remove it. Political and engineering strategies are suggested, including identifying problems, setting levels of protection, and reducing hazards. The division of responsibility among government, developers, and homeowners is conceptualized. The future impacts of public works, the National Flood Insurance Program, and floodplain management by local agencies are evaluated.

BACKGROUND

Nature of the Problem

Sediment-laden floodflow has caused millions of dollars worth of damage in the Southwest during the last decade. Almost every year some community having an annual rainfall of less than 20 in. is struck by devastating flash floods of water, rock, sand, and mud. The watercourse producing the damaging mudflows is dry most of the time. The concentration of solids in the flow may range up to 50 percent.

To most Americans the word "flood" evokes images of the Johnstown flood, Hurricane Agnes, or the unruly Missouri River sweeping out of its banks through the Kansas City stockyards. Television newscasts of "eastern" type floods illustrate broad areas covered with standing water, people and livestock clustered on roofs, and rowboats carrying people to safety. In the arid Southwest there is a completely different and special hydrologic event: the sediment-laden flood.

John M. Tettemer is Acting Chief Deputy Engineer with the Los Angeles County Flood Control District in Los Angeles, California.

Sediment flows are part of the natural process by which mountains are worn away and washed toward the sea. Over geologic time the sand, rock, and soil eroded from the mountains have been laid down at their feet in broad, sloping, cone- or fan-shaped deposits. The entire surface of these "alluvial cones" is laced with abandoned gullies and stream channels. At one time or another, sediment-laden flows have traversed all these routes. Flow paths are unpredictable. Each flood causes sediment deposits, which obstruct the channel and send the next flow in a new direction. Over the centuries, imperceptibly, the cones grow larger and the mountains grow smaller.

After the cone has been urbanized, as it has in Las Vegas, Palm Springs, Pasadena, and many other southwestern sun belt communities, the "imperceptible" growth rate can suddenly become unmistakably apparent. The flash sediment flood on an urbanized cone is an unforgettable experience, involving the battering and destruction of homes and automobiles and the filling of houses with sediments.

Factors Involved

Sediment floods occur in the southwestern United States: West Texas, New Mexico, Arizona, southern California, Nevada, and Utah. Several ingredients are common to communities subject to this hazard. The first is an arid climate. Rainfall ranges from 3-1/2 in. per year to 20 in. In the drier interior regions vegetation is sparse. Desert soils are not protected by a canopy of leaves that can absorb the impact of rainfall. Ground cover and roots are not present to stabilize the soil against erosion. In the wetter coastal regions the vegetation is hardy chaparral brush that can survive months of drought but be destroyed in a few hours by brushfire. With the protective cover burned away, the delicate soils are exposed to erosion.

The next ingredient is intense rainfall. Coastal regions experience large slow-moving cyclonic winter rainstorms originating in the northern Pacific Ocean. Sediment production is maximized when, after several days of saturating rainfall, frontal passage is accompanied by extremely high-intensity rainfall. Raindrop impact, overland flow, and gully erosion can strip tons of sediment from an acre of land in a few minutes. In the interior deserts the force of the Pacific winter storms has usually abated by the time they arrive, so they do not present the most critical threat of erosion. Here it is the tropical storms and convective thundershowers that present a threat. Tropical storms originate in the Gulf of Mexico, the Gulf of California, and the Pacific Ocean and occur most frequently from July through September. Summer thunderstorms result from the heating, convergence, orographic lifting, or frontal lifting of moist air passing through the region. The most dangerous flood-producing storm is one covering an area of about 400 square miles, lasting about three hours, and characterized by intense downpouring of rainfall.

Erosive soils constitute the next ingredient. In California the formation and adjustment of the mountains by continental plate movement and earthquakes have left the mountains in a fractured, pulverized condition. Mountainsides have thin soil layers that erode or slide easily in heavy rain. Also, alluvial cones are often not well cemented and are unstable when erosion starts. Brushfires on coastal hillsides glaze the surface, increasing the runoff until rills and gullies form and soil begins to move. Wind-deposited soils in the deserts are fine grained and noncohesive. All of these situations contribute to scouring and gullying, producing a flow of liquid mud and rock that can be carried downhill with destructive force.

Another troublesome factor is a change of slope, from steeper to flatter, as elevation decreases. The higher, steeper areas often receive more rainfall, and the steepness promotes high-velocity runoff and erosion. The steep canyons concentrate high-velocity flows, providing the energy to move rock and mud rapidly downstream. When the flow emerges from the steep canyon and strikes the flatter alluvial cone, it loses some of its kinetic energy and its ability to keep sediment moving. The rock and mud drop out of the flow, clogging channels, filling streets, yards, and houses, and building up the ground level for the next flow.

Brushfires are an extremely important factor in those areas having sufficient rainfall to support chaparral. With its compact canopy and tenacious roots the chaparral community of plants provides good protection for the fragile soils on steep southern California hillsides. After a burn the dry soils are so unstable they may run like hourglass sand, collecting at the bottoms of slopes in cones. Of all erosive soil conditions the steep burned chaparral hillside seems to be the worst.

Finally, there is the ingredient that makes the natural phenomena listed above such costly problems: urbanization. The alluvial cones, with their breathtaking views of city lights below and mountain ranges in the distance, and the wooded canyons surrounded by chaparral hillsides, make up the suburban fringe that has been undergoing extensive development during the last 20 years. The sites have prestige and command high prices. For the most part, the development in place today was not designed with adequate recognition of the sediment flow hazard to which it is exposed.

Size of the Problem

The problem of damge from sediment flow is serious not so much because vast areas are involved but because high-value improvements are concentrated within relatively narrow areas. Of the 500,000-square-mile area comprising the nation's Southwest, about one half is mountainous and mostly in federal ownership. Another 200,000 square miles is desert, much of which is federal land. Only 50,000 square miles is river valley land, with agriculture and cities. The areas affected by sediment flow are the sloping alluvial plains located below the mountain ranges. Such plains make up about 40,000 square miles of land. In 1979 it was estimated that only about 1,000 square miles of these plains are completely or partially urbanized. About 2,500,000 structures are involved, with a replacement value of $100 billion.

The cost of damages increases yearly as human occupancy expands. The Las Vegas sediment flood of July 3, 1975, caused $4 million worth of damages. Tropical storm Kathleen, in September 1976, caused $23 million worth of flood and sediment damage in the City of Palm Desert, California, plus additional millions to utility, transportation, and communications facilities. The same area was struck again in 1979, with damages estimated at $50 million. Sediment flow during February and March 1978 in Los Angeles caused $100 million worth of damage to private and public structures, roads, utilities, and flood control works (U.S. Army Corps of Engineers, 1978). It is estimated that the total average annual cost of sediment-related flood damages in the Southwest is $20 million in 1979 dollars. These figures could double in the next 10 years unless development policies and criteria are modified to recognize and mitigate sediment hazards.

Impacts

Individuals

The most tragic impact of sediment floods is upon individuals and families. Unlike riverine flooding, sediment flooding leaves behind crushed stucco walls, thousands of tons of rock, mud, and debris, and automobiles hammered around standing trees. The destruction and the cleanup problems are staggering. Homes representing a family's major financial asset are destroyed.* Livestock and pets are lost forever. One father leading the family horse to safety across a stream was swept away and drowned. Others have been stricken with heart attacks while shoveling mud and lifting sandbags. Highway workers placing barricades in front of a dip crossing of a normally dry stream have been swept away and drowned. Caretaker residents of a church camp resort area have been swept away in the night. Bodies not yet recovered are probably buried in downstream reservoir sediments.

Community

At the community level the sediment flood brings several reactions. During the emergency the shock of the disaster stimulates heroic and unselfish acts of courage and strength. Natural leaders take charge of evacuations, seeing to it that the elderly, disabled, and young are carried to safety. Strapping teenage boys and girls set up sandbag operations, filling and placing the heavy bags long past the point of exhaustion. Emergency forces from every source--utilities, contractors, and public agencies--all find ways to contribute without concern for jurisdiction or red tape.

The morning after the flood, when the shock has worn off, residents walk the streets and canyons, surveying the wreckage. As they look at the path of destruction leading from canyon mouths and spreading randomly across alluvial cones, they realize how certain it was that disaster would strike where it did. After the flood it does not take an expert to observe that mudflows that have always poured out of the canyons will continue to occur, even if houses are placed in their way. As people gather, the question is asked over and

---

*At this writing almost none of the structures damaged and destroyed have been insured, although federal flood insurance has been available since 1968.

over: "How did they ever approve a building permit in that spot?" The feeling of betrayal by the officials entrusted with public safety spreads rapidly under these circumstances. At a time when distrust of government is already at a high level, we do not need to add fuel to the fire or increase public liabilities. Past practices governing the development of sediment hazard areas are sure to leave the community with a feeling of bitterness, resentment, and betrayal toward agencies that regulate development in the name of public safety.

Local Government

The impact of sediment floods on local government is staggering in terms of costs of restoration, interruption of services, and regulatory and political quandries. In one southern California flood, damages to public properties, roads, bridges, and flood control facilities amounted to $84 million. Although restoration of public facilities is eligible for federal disaster funding under certain circumstances, it takes about 60 days to receive the first payment. Local agencies must be able to finance emergency flood fighting activities during the interim, which may involve enormous cash outlays for rental equipment, operators, and contracts.

Diversion of local financing to flood fighting and restoration operations means deferring other projects and services that were scheduled and financed. Energy and money expended on restoring public facilities damaged by sediment floods can never be recovered and are permanently lost to society.

Repeated sediment floods have had an impact on the way local officials view their responsibilities for controlling development. At the technical level attention has been drawn to the engineering aspects of sediment flooding. Improved procedures for evaluating proposed development, for predicting the quantity and location of potential sediment flows, and for setting criteria to mitigate their hazard have been developed. Planning, zoning, subdivision, and building departments have become much more aware of sediment hazards and receptive to procedures and techniques for avoiding or mitigating them. Although only a few communities have taken positive mitigating steps, they have demonstrated that workable procedures and criteria can be developed and implemented without upsetting the housing industry.

At the political level the same thing has happened. Immediate reactions have included ordering reevaluations of planning and building criteria. Beneficial results have included a good understanding of the seriousness of sediment hazards and a willingness to stand behind the recommendations of technical staff on safety criteria. Again, this political perspective is not widespread, but it does indicate that concerned engineering officials can work effectively with elected officials to improve public safety.

Federal and State Government

Disaster relief laws authorize the federal government to assume most of the cost of restoration of public facilities whenever the President declares a national disaster. Repair and restoration of public buildings, streets, parks, and flood control facilities due to sediment flood damage cost an estimated $10 million in southern California. States are also impacted, particularly when the situation does not qualify for a proclamation of national disaster. Since states do not normally maintain an appropriation for this purpose, it is usually necessary to enact special assistance legislation. The overall effort amounts to a substantial deployment of energy and money that the nation can ill afford.

ENGINEERING ASPECTS

Factors Affecting Sediment Production

Sediment flow as used in this paper means a high concentration of rock, sand, and soil in floodwater such that great destructive force is generated. Sediment production rates for a single storm have been measured as high as 240,000 cu yd per square mile. At peak sediment flow rates it is estimated that the sediments constitute up to one half the total volume of flow. The largest body of quantitative engineering data on sediment production is contained in the records of the Los Angeles County Flood Control District.

The district has several flood control reservoirs with 50 years of record. Of the 100 debris basins presently operated, 18 have more than 36 years of record, and 25 have at least 25 years of record. It has generally been believed that sediment production is a function of watershed variables such as soil types, size of drainage area, steepness, vegetative cover, rainfall, fire frequency, aspect, and relief ratio. Recent regression analyses have indicated that sediment production rates per unit of watershed area are most influenced by the vegetative cover (which is a measure of fire history), followed by relief ratio and rainfall. The other factors measured did not significantly improve the results. No practical way has been found to introduce soil and geology data into the regression analysis.

Sediment Volume Prediction Models

For many years the debris basin design standards of the Los Angeles County Flood Control District relied on enveloping curves based on historical sediment measurements. By 1956 sufficient data had been accumulated to perform regression analyses using watershed variables (Los Angeles County Flood Control District, 1959). In 1979 a new regression analysis was performed for the Federal Insurance Administration of the U.S. Department of Housing and Urban Development using a much expanded data base. It had as a specific objective the establishment of a frequency basis so that flood insurance rates for mudflows might be calculated (Los Angeles County Flood Control District, 1979). The procedure required (1) determining an appropriate frequency distribution that would fit historical data and extrapolate to reasonable values, (2) relating sediment production to measurable watershed parameters for estimating sediment production from ungaged watersheds, and (3) properly accounting for the effect of fires in expected sediment production. (See Los Angeles County Flood Control District (1979) for details.)

Hydraulics of Sediment Flow

The hydraulics of mud and sediment flow present problems. Hydraulic equations used for water cannot be fully relied upon because the density and viscosity of sediment flows are so different from those of water. Flow records are scarce because gaging stations that receive sediment-laden flow are often buried by the event for which the record is desired. Poststorm observations using the slope-area method are questionable because of flow density and viscosity problems. Accurate velocity measurements are hard to get because of the difficulty of operating a current meter in sediment-carrying streams. It has been observed that sediment-laden floodflows often arrive in tremendous surges or waves many times the average flow rate. The effects of this slug flow are several.

1. Instantaneous flow rates and velocities are much higher than one might expect.
2. The depth of flow and corresponding damage potential are higher than one might expect.
3. The tremendous kinetic energy can destroy structures on impact and can overwhelm a debris basin designed for a steady inflow rate. Reliable postflood measurements of sediments that passed debris basins are nearly impossible to make.

The evaluation of sediment hazard potential requires the ability to predict the location and amount of deposition, because the recession flows will be passing over the deposited material. Where deposits have occurred in the past, there is a basis for estimating future deposition and for extrapolating the relationships to other locations. Factors that influence deposition are those that influence velocity: slope and cross section. Changes of slope and obstructions caused by walls, fills, automobiles, and houses are the main indicators of sediment deposition.

While the hydraulic flow characteristics of sediment-laden flows present some technical problems to the engineer, adequate representation of the potential hazard can be made by approximate methods. This enables the identification of hazardous areas and the design of mitigation measures, both nonstructural and structural. It is more important to recognize the hazard and apply a commonsense approach than to worry about the precise width and depth of the flow.

Debris basin capacity has come under new scrutiny recently. As a result of heavy rains in 1978 following a large local brushfire after two years of drought, erosion and sediment production were maximized. Watersheds had been saturated by intermittent showers for several weeks. In Zachau Canyon high-intensity rainfall produced surges of sediment-laden flow containing 10-ton boulders. The flood crest hit debris basins in a wave, overrunning the structures and sending boulders and mud downstream. Boulders and chain link fence from the channel walls plugged underground channels downstream, and communities were devastated by boulders and mud. This event demonstrated that under conditions marked by freshly burned watershed the previous understanding of sediment deposition, based on experience, was inadequate. The dynamics of the sediment-laden mass entering basins obviously governed their performance. The data gathered during this flood will be evaluated to establish the determining relationships.

CONTROLS

Keep it Moving

Engineers have only two choices with sediment flow: either stop it or keep it moving. Where it is physically possible to keep it moving without causing damage, there are advantages. If the stream is a coastal stream, beach starvation can be avoided by allowing sediment to pass to the ocean. Also, if the sediment can be carried downstream by the water, it will not have to be cleaned out of expensive sediment containment structures and carried away at substantial cost.

There are problems with and limitations to the concept of improved sediment-carrying channels. There may be insufficient slope to keep sediment moving. With insufficient slope the channel will plug and the community will be flooded. Where this is the case, there is no choice but to trap the sediment. Where there is sufficient channel slope to carry sediment to a safe destination, the channel will perform satisfactorily but will wear out due to abrasion of the channel's concrete bottom. After a few years the steel will be exposed and the channel bottom will have to be relined.

Situations where sediment transport will work include streams above existing debris basins or reservoirs with enough capacity for the sediment. Also, small sediment-carrying side canyons may be introduced into a large channel, where the main channel flow will always be large enough to handle the sediment load. It is also possible to design a sediment-carrying floodway with a natural bottom and revetted levees. Such a floodway may act as a linear sediment basin. Adequate freeboard must be provided to contain the deposition, and arrangements must be made to excavate aggrading deposits to restore channel capacity.

Sediment-carrying channels should be designed for bulked flow, should be free from grade breaks that flatten the grade, should be free from restrictions and expansions, and should be as straight as possible. The channel should be open for ease of maintenance. Trapezoidal and V sections provide better scouring velocities at low discharges than do rectangular channels. Invert concrete should be extra thick to allow for abrasion, and scour gages consisting of colored concrete cones should be imbedded in the invert so that wear can be monitored. Studies show that concrete is the least expensive material at this time.

Restrictions and expansions are hard to avoid in stream systems undergoing urbanization. Sections of leveed channel may alternate with natural floodplains. Special care is needed to maintain sediment transport.

Separation of Sediment from Water

If the sediment-laden flow cannot be safely carried through the community to a safe point of discharge, it will be necessary to remove the sediments and carry only the water. Sediment removal is accomplished by providing a reservoir or basin large enough to contain the sediment from at least one major flood. A spillway is provided to protect the embankment from overflow. A drain should be provided to dewater the basin to simplify cleanout operations. In areas having high sediment rates the basin capacity measured to the spillway crest may be as much as 240,000 cu yd per square mile of drainage area (equivalent to a depth of 0.23 ft or 7 cm over the watershed area). Sediment tends to deposit in the basin on a slope, so basin capacity is sometimes computed with a plane extending upstream from the spillway crest at a slope one-half that of the natural streambed. Sediment basins are usually located in populated areas, where it is necessary to fence them for security. The basins must be cleaned out after storms, so all-weather access is necessary. A sediment disposal site suitable for the long term must be provided as near as possible, which may be difficult where land value and environmental concerns stir a strong public response. The site must be planned with regard to the stability of the fill, access, drainage, and truck traffic during cleanout operations. Community public relations during prolonged flood fighting periods when trucks are operating day and night can become a significant effort. Typical design features of a sediment basin are shown in Figure 1.

Sediment basins are high-cost, high-maintenance facilities that can provide a high level of protection. They are justified only in situations where land use requirements and real estate values preclude leaving the hazardous area open for sediment flows. Most sediment basins are built to protect existing communities located in hazardous areas. New developments can often be designed to occupy safer ground and leave the sediment-carrying canyons alone, if adequate information on sediment flow hazards is available during the planning stage.

POLITICAL-ENGINEERING STRATEGIES

Problem Identification

The sediment flow hazard is not well understood by many public works engineers or public officials. As understanding of the relationship of sediment flow to other hydraulic considerations in flood control improves, it will be possible for governmental agencies to identify specific problems from drainage area to drainage area. Clear understanding of sediment flow by public works officials will increase confrontations with land developers and therefore with political leadership interested in healthy growth. Some agencies are already mapping areas where sediment flow is a consideration. This will be seen on zoning and other master-plan documents. This is a first step; it provides notice to prospective buyers and elected officials that a special problem exists. It is the responsibility of public works officials to

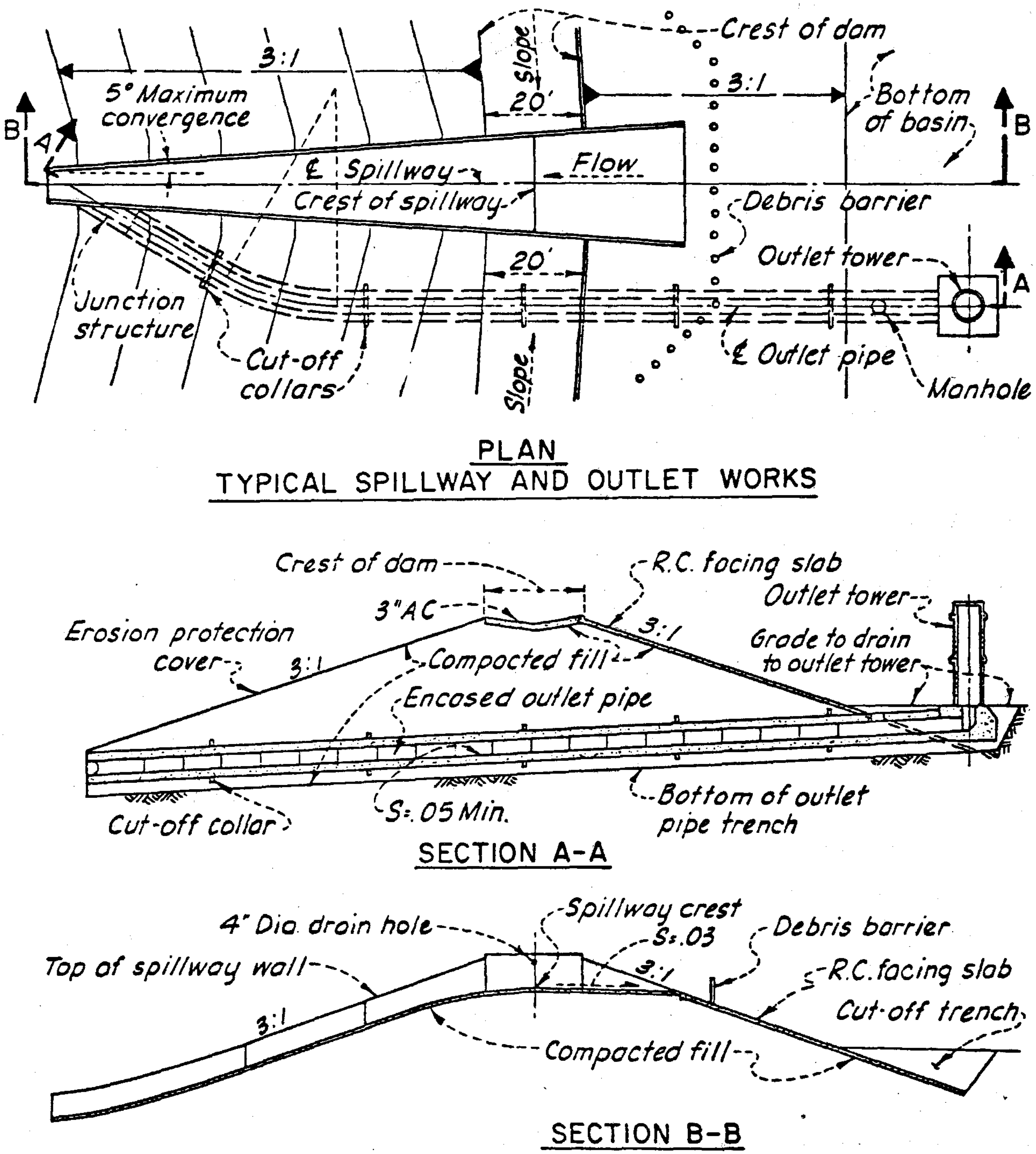


FIGURE 1 Typical sediment basin configuration.

obtain sufficient knowledge to allow the controlling public agencies to adopt land use management plans that consider sediment flow. National Flood Insurance Program (NFIP) mudflow mapping will go a long way in this direction.

100-Year Versus Other Standards

The NFIP mapping and regulatory standard basis of 100-year clear water and 100-year mudflow hazards has serious limitations in urban areas. Greater flows have a demonstrated record of doing major damage. It is probable that this flood protection level may have to be increased in some areas as more statistics on loss become available. Some flood control agencies use standards considerably higher than the 100-year standard when designing facilities or establishing floodplain management techniques for high-density urban development. Each agency should knowingly adopt a flood protection standard and develop strategies that reflect that standard for floodplain management. If the floodplain will be relatively sparsely developed, the 100-year protection level may be adequate, if structures are raised and properly located. For higher-density urban property the 100-year protection level may be inadequate, and sediment flows may cause extensive damage. This is a local problem that should be resolved by local floodplain management regulations. It is important that political bodies take specific actions to establish a level of protection for sediment flows, since they will be called upon to provide public assistance when the NFIP standards are exceeded, and since they and their employees may incur total liability for either inaction or negligent actions. Public works engineers need to educate elected officials about these realities even though sediment flows may occur relatively infrequently.

Hazard Reduction

The objective of floodplain management is to reduce hazards and economic losses. The reduction of hazards has political overtones, since land use may be restricted or costs may be increased by flood control structures. Engineers bear the responsibility to develop alternative proposals that are consistent with their drainage and sediment flow master plans. These proposals must be communicated to political bodies concisely and clearly so that the regulations and related land use or construction costs are understood. Without such understanding, political bodies are prone to approve individual developments because they do not each have a significant impact, without recognizing that on a long-term cumulative basis they may be thwarting the master plan and committing the community to enormous remedial costs, or perhaps setting the stage for a future flooding catastrophe.

Governmental Obligations

Initiatives by the federal government are motivating local governments to regulate hazardous areas. The significance of this is apparent when one reflects on the decades of laissez-faire developmental policies by local governments. Although it is too early to evaluate the flood insurance program, indications are that communities are ready to regulate hazardous areas if they have the necessary information and the political message that they have no choice under the federal mandates.

One important element in a comprehensive solution is an obligation of the federal government that has not yet been implemented. Section 1362 of the National Flood Insurance Act of 1968 as amended authorizes federal purchase of severely flood-damaged properties provided the properties are insured under NFIP, the local building department will not allow their reconstruction, and a local elected official requests the action. Until this element of the program is implemented, there is no mechanism for breaking the cycle of damage, repair, and resale. Homes in hazardous areas have been struck as many as four times by sediment floods, suffered each time by new owners. Local governments now must proceed with care so that their programs go beyond minimum federal requirements and meet their own long-term needs.

Developers' Obligations

The key to sound development is not governmental regulations but practical understanding and resolution of the sediment and flood problems by developers. When sediment flood hazards have been identified at an early stage, key decisions can be affected--such as whether to purchase one parcel or another, and what type of development to plan. During development planning the grading concept, street layout, and lot designs can all be adjusted to accommodate and mitigate sediment hazard. Finally, dwelling placement and orientation on the lot, as well as construction and elevation, should reflect awareness of sediment hazard. The basic rules are as follows.

1. Determine the flood and sediment hazard to each lot.
2. Provide a safe pathway for sediment-laden flow to a safe point of discharge, without harm to structures and improvements.
3. Do not flatten the grade of sediment flow paths or try to change their alignments.
4. If a drainage facility is necessary, design it as an open channel so that it can be cleaned out.
5. Allow equipment access for cleanup.
6. If sediment cannot be safely carried through on the surface, provide an adequate basin, with access for maintenance.

Homeowner Obligations

Homeowners must understand the problem so as to mitigate it within their means or not make it worse. They must particularly be aware of the sediment flow path and the room it requires. This path must be left clear, so block walls, accessory buildings, and landscaping should be planned accordingly. Homeowners can also improve their safety by constructing properly designed deflector walls to keep sediment-laden flow moving in the right direction. Finally, they should protect against loss by taking out flood insurance.

Insurance Aspects

The sediment hazard to many existing developments is so diffuse that it cannot be cured by a major flood control project. The only recourses available to owners of such properties are to maintain and improve sediment flow paths as well as they can and take out insurance. The insurance will defray the cost to individuals of restoring damaged structures and will significantly reduce the impact on family finances. On a longer-term basis the flood insurance program requires a buy-out when the structure is damaged by more than one-half its value. This provision will eventually eliminate many existing hazards by allowing hazardous lots to revert to open space or safe uses.

THE FUTURE

The Public Works Construction Outlook

Public works construction (including flood control facilities) accounted for a significant portion of federal, state, and local expenditures during the 1960s. Since that time rising costs and changing priorities, together with environmental awareness, have markedly reduced the amount of money available for public works. Now that the nation has entered an era of insufficient energy and escalating costs, it appears that there will be increasing competition for the nation's scarce dollars. New public works construction will probably receive relatively low priority within the next generation. Consequently, people should not expect that even the existing flood and sediment hazards can be cured. There is certainly no reason to expect that public works will rescue new developments placed in unsafe areas. Local planners and elected officials must realize this and shift gears from the old viewpoint that the U.S. Army Corps of Engineers or the local flood control district will correct any problems caused by unwise development.

Impact of Federal Flood Insurance

Despite some recognized shortcomings, the flood insurance program is still the only game in town in terms of an organized effort to focus the attention of local officials on their responsibilities for safe development. With the flood insurance program as a catalyst, local floodplain management programs can be developed that will significantly reduce future sediment damages. Local agencies can set standards of protection commensurate with their community's plans, so long as they at least satisfy federal requirements. Mechanisms for implementation can vary, depending on local philosophy and existing ordinances. Most importantly, the effects of future development can be reflected in hazard calculations and be kept consistent with the community's own goals. Eventually, the need for the government to purchase severely damaged properties will be recognized. Implementation of this program will go far toward eliminating the most serious existing hazards. The flood insurance maps may provide the stimulus needed by communities to support public works flood control programs. When mandatory flood insurance is in full effect, property owners may find they would be better off to support a bond issue to eliminate the mandatory insurance than to continue paying the premiums. In any event, the flood insurance program does offer better financial protection for property owners in flood hazard areas, better public information about flood hazards, and better regulation of new development.

The Impact of Local Floodplain Management

Pressure to develop new land will continue. New lands generally are those that either have been passed over before or have not been reached by development. Either way, it often turns out that the land available for development has problems, such as a sediment hazard. Recognizing and mitigating the hazard will tend to increase the cost of development, so floodplain management is initially viewed in an unpopular light by developers. In the long run, if the criteria are reasonable and fairly applied by all jurisdictions in an area, the housing industry accepts them as part of the cost of providing quality housing. There is no question but that the widespread adoption of the measures to mitigate sediment hazards described earlier in this paper will enormously affect the future loss of life and property to sediment flood-related disasters.

REFERENCES

Los Angeles County Flood Control District (1959) Report on Debris Reduction Studies for Mountain Watershed of Los Angeles County, Los Angeles County Flood Control District, Los Angeles.

Los Angeles County Flood Control District (1979) Mudflow Methodology, Los Angeles County Flood Control District, Los Angeles.

U.S. Army Corps of Engineers (1978) Floods of February and March 1978 in Southern California, U.S. Army Corps of Engineers, Los Angeles District, Los Angeles.

THE STORMS OF 1978 AND 1980 AND THEIR EFFECT ON SEDIMENT MOVEMENT IN THE EASTERN SAN GABRIEL FRONT

by Wade G. Wells II

The effects of three major storms, one occurring in 1980 and two in 1978, are compared using data from the U.S. Forest Service's San Dimas Experimental Forest. The San Dimas Experimental Forest is located near Los Angeles, California, in the eastern San Gabriel Mountains. The 1980 storm, although it was the largest ever recorded on the experimental forest, seemed to be less destructive than the 1978 storms. Sediment measurements from debris basins tended to support this observation.

Sediment production from two study watersheds on the experimental forest was about 40 percent less in 1980 than in 1978. Although there may have been several reasons for this, it is probable that there simply was not as much sediment available for movement in 1980 as there was in 1978. The same channels that were scoured by the 1980 storm were also heavily scoured by the storms in 1978. With only two intervening years for fresh sediment accumulation, these channels may still have been relatively clean when the 1980 storm occurred.

Fires play a more important role in sediment production than do even the most severe storms. Sediment production from freshly burned catchments during relatively minor storms frequently exceeds that from the most severe storms. The worst sedimentation events occur, of course, when a severe storm strikes a recently burned watershed, and examples of this are presented.

INTRODUCTION

The storm of February 13-21, 1980, is the largest single storm ever recorded at the U.S. Forest Service's San Dimas Experimental Forest in the eastern San Gabriel Mountains. During this storm 628 mm of rain was recorded at the Tanbark Flat rain gage (elevation 825 m), and 458 mm was recorded at the Glendora Ranger Station (elevation 252 m) 10 km southwest of Tanbark Flat

---

Wade G. Wells II is Research Hydrologist with the U.S. Forest Service at the Pacific Southwest Forest and Range Experiment Station in Glendora, California.

(see Figure 1). When compared with other storms of a similar magnitude, the amount of sediment produced by this storm was unusually low. There are also indications that property damage from this storm was not as high as one might expect.

HISTORICAL PERSPECTIVE

Records have been kept at Tanbark Flat since October 1928, and during this 51-year period four unusually severe storms have occurred. Table 1 lists the dates of these storms along with their total rainfall at both Tanbark Flat and the Glendora Ranger Station.

TABLE 1 Unusually Severe Storms at Tanbark Flat

	Total Rainfall (mm)	
Storm Dates	Tanbark Flat	Glendora
Major storms		
Feb. 27-Mar. 3, 1938	538	343
Jan. 23-26, 1969	467	232
Feb. 27-Mar. 4, 1978	419	324
Feb. 13-21, 1980	628	458
Lesser related storms		
Jan. 18-22, 1969	335	207
Feb. 5-10, 1978	302	174

Before the storm of 1938 the last major storm appears to be that of January 15-18, 1916. This storm produced 344 mm of rainfall at the 100-year-old West gage in Glendora. This gage, the second oldest active gage in Los Angeles County, consistently gives readings within 2 percent of the Forest Service gage. It can be concluded, therefore, that the 1916 storm and the 1938 storm were very similar.

Two other storms are also listed in Table 1 because of their close relationship to the four major storms. The first occurred on January 18-22, 1969, and dropped 335 mm of rain at Tanbark and 207 mm at Glendora. This resulted in a nine-day total of 802 mm at Tanbark, the wettest such period on record. The second storm occurred on February 5-10, 1978, and does not rank as a major storm for the eastern San Gabriels. It produced only 302 mm of rain at Tanbark and 174 mm at Glendora. It caused extremely high flows, however, and moved unusually large amounts of sediment for its size.

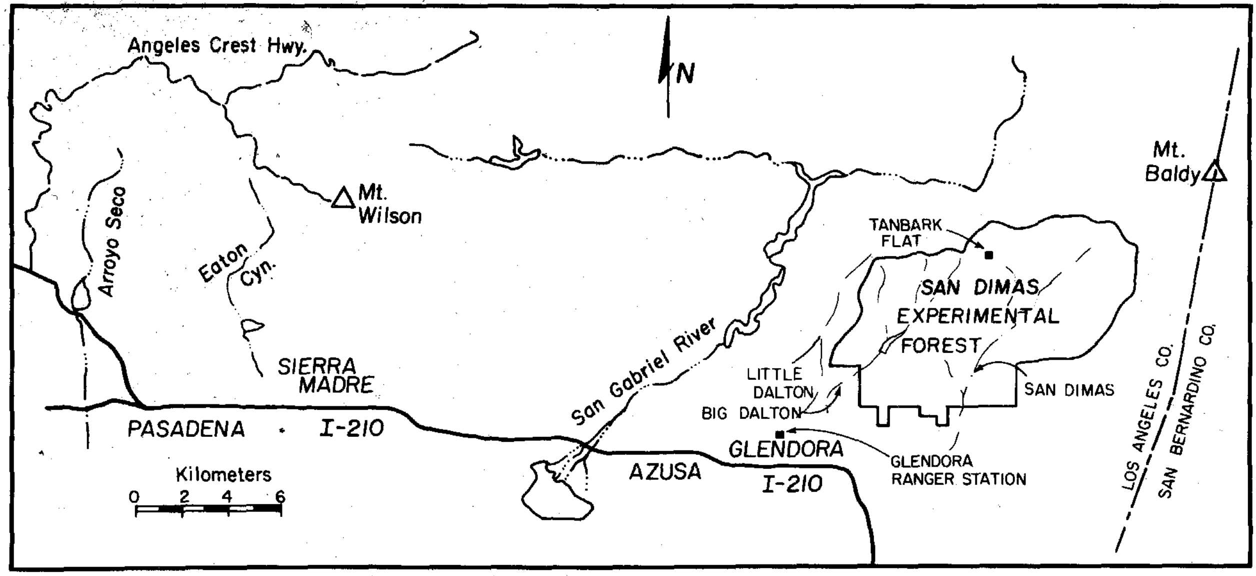


FIGURE 1 Map of eastern San Gabriel front between Arroyo Seco and the Los Angeles-San Bernardino county line.

This report compares the storm of February 13-21, 1980, with those of February 5-10 and February 27-March 4, 1978, by looking at their effects on two small watersheds in the San Dimas Experimental Forest. It is not always possible to separate the effects of the two 1978 storms, but, when possible, this has been done. The early storm showed unusually high flows and sedimentation rates, but the later storm apparently did more total damage.

DESCRIPTION OF THE STUDY WATERSHEDS

The study watersheds are located at the head of Bell Canyon, a tributary to Big Dalton Canyon, about 2 km west of the Tanbark rain gage in the San Dimas Experimental Forest (see Figure 2). These catchments, known as Bell 2 and Bell 3, have been monitored for streamflow and sediment production since 1933. Bell 2 is a 40-ha south-facing watershed that is covered with a mixture of California buckwheat, artificially seeded annual grasses, and occasional clumps of chaparral. Bell 3 is 25 ha in size, faces southeast, and is covered by native chaparral. The average slope for both watersheds is around 65 percent, and their elevations range from 760 m to 1,060 m in Bell 2 and from 760 m to 1,030 m in Bell 3. Both catchments burned in 1919 and 1960. Bell 2 also burned in 1975, but Bell 3 did not. Sediment troughs were installed on the slopes of Bell 2 after the fire to monitor postfire debris production, and these were maintained until early 1979.

Sediment is trapped in small concrete-lined debris basins, and streamflow is measured by 120-degree V-notched weirs placed below the basins (see Figure 3). Channels are steep and well armored, with significant reaches lying directly on bedrock. Cascades and small waterfalls are numerous.

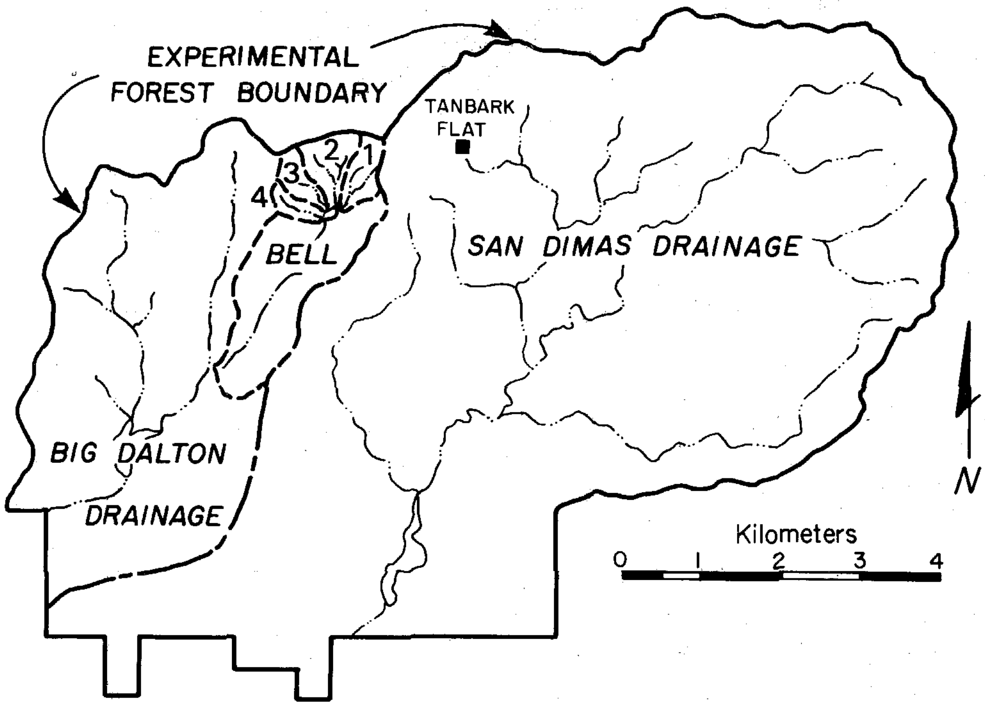


FIGURE 2 Map of the San Dimas Experimental Forest showing location of Tanbark Flat and the Bell watersheds.

STORM CHARACTERISTICS

Hyetographs for each storm, as recorded at Tanbark Flat, are shown in Figure 4. Each bar of the hyetograph represents three hours of rainfall. It is obvious that the first four days of the 1980 storm were much more severe than either of the 1978 storms. During this four-day period 463 mm of rain fell (73 percent of the total), and 204 mm (44 percent of the total) fell on a single day, February 16, 1980.

Comparing the 1980 storm with the 1978 storms, we see that the heaviest rainfall occurred on the fourth or fifth day of each storm. Also, antecedent moisture conditions for all three storms are roughly equivalent. There were 12 rainless days preceding the storm of 1980, 12 rainless days before the storm of February 27-March 4, 1978, and 15 rainless days out of 16 before the storm of February 5-10, 1978. It is possible, however, that the heaviest rain in the storm of February 5-10, 1978, did not fall on a fully saturated watershed. During the 3-1/2 days preceding its period of heaviest precipitation, only 57 mm of rain fell. In the other two storms the periods of heaviest rain each came after over 200 mm had fallen.

Differences are also found in the peak intensities of each storm. Figure 5 shows the peak intensities of each storm for various time periods as recorded by the gage at Tanbark Flat. The storm of February 27-March 4, 1978, had the highest short-term (less than one hour) intensities, but the 1980

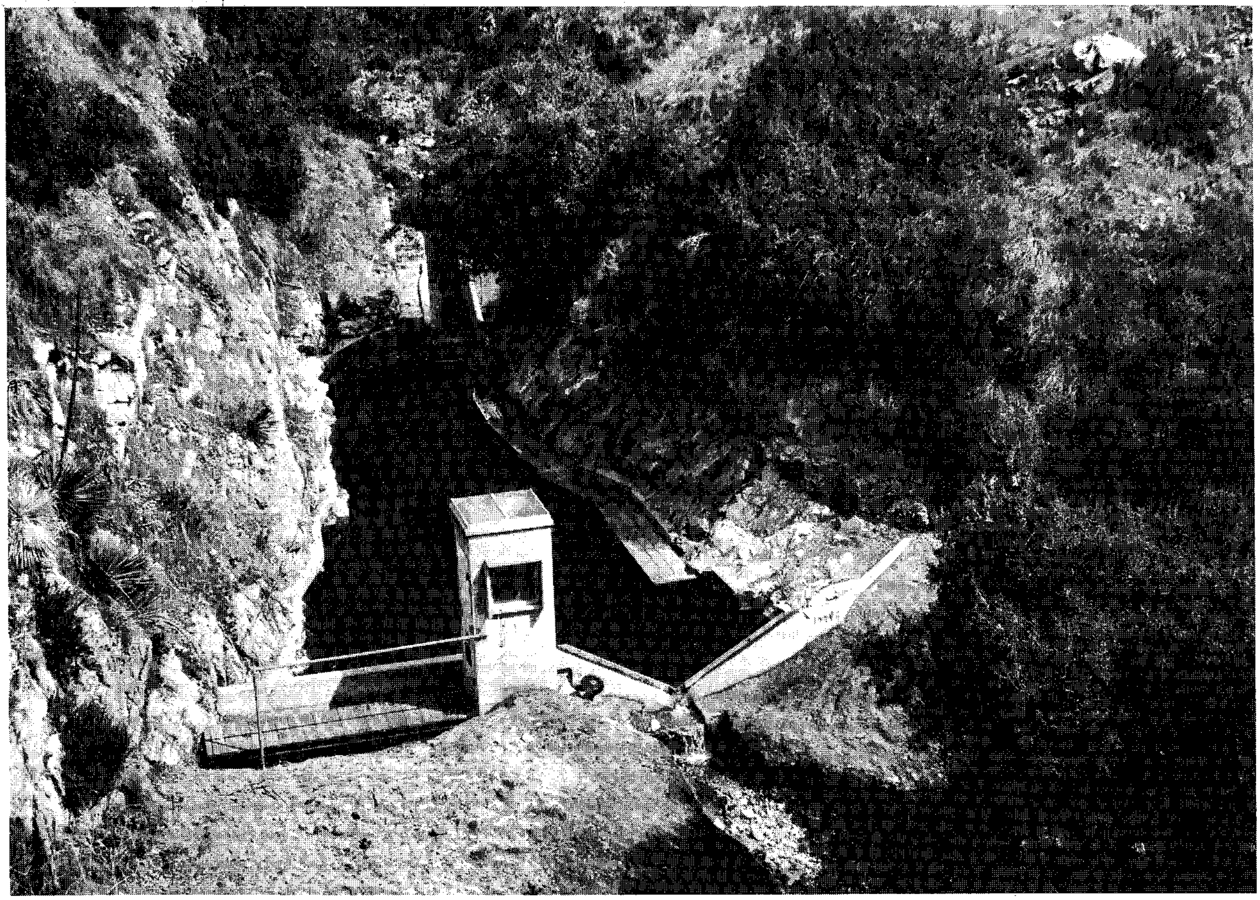


FIGURE 3 Bell 2 debris basin. Note concrete approach at upper end and well-armored channel above it.

storm shows much higher long-term intensities. In fact, on February 16, 1980, it rained at over 30 mm per hour for five consecutive hours. The storm of February 5-10, 1978, was milder in all respects than the other two.

SEDIMENTATION DURING THE 1978 AND 1980 STORMS

Table 2 shows sediment production from both catchments for the storms of 1978 and 1980. The debris basin in Bell 2 filled, but did not overtop, during the storm of February 5-10, 1978. It was not possible to clean out the basin before the storm of February 27-March 4, 1978, occurred. Therefore no data for the later storm are available from Bell 2. The Bell 3 debris basin was not measured between the two 1978 storms, so the sediment production data present from this catchment are for both 1978 storms. Observers estimated that over two thirds of the 1978 debris production in Bell 3 came during the February 5-10 storm, but there are no actual measurements to confirm this.

Two items in Table 2 are notable. First, peak flows from the storms of February 5-10, 1978, and February 13-21, 1980, are roughly equivalent, despite

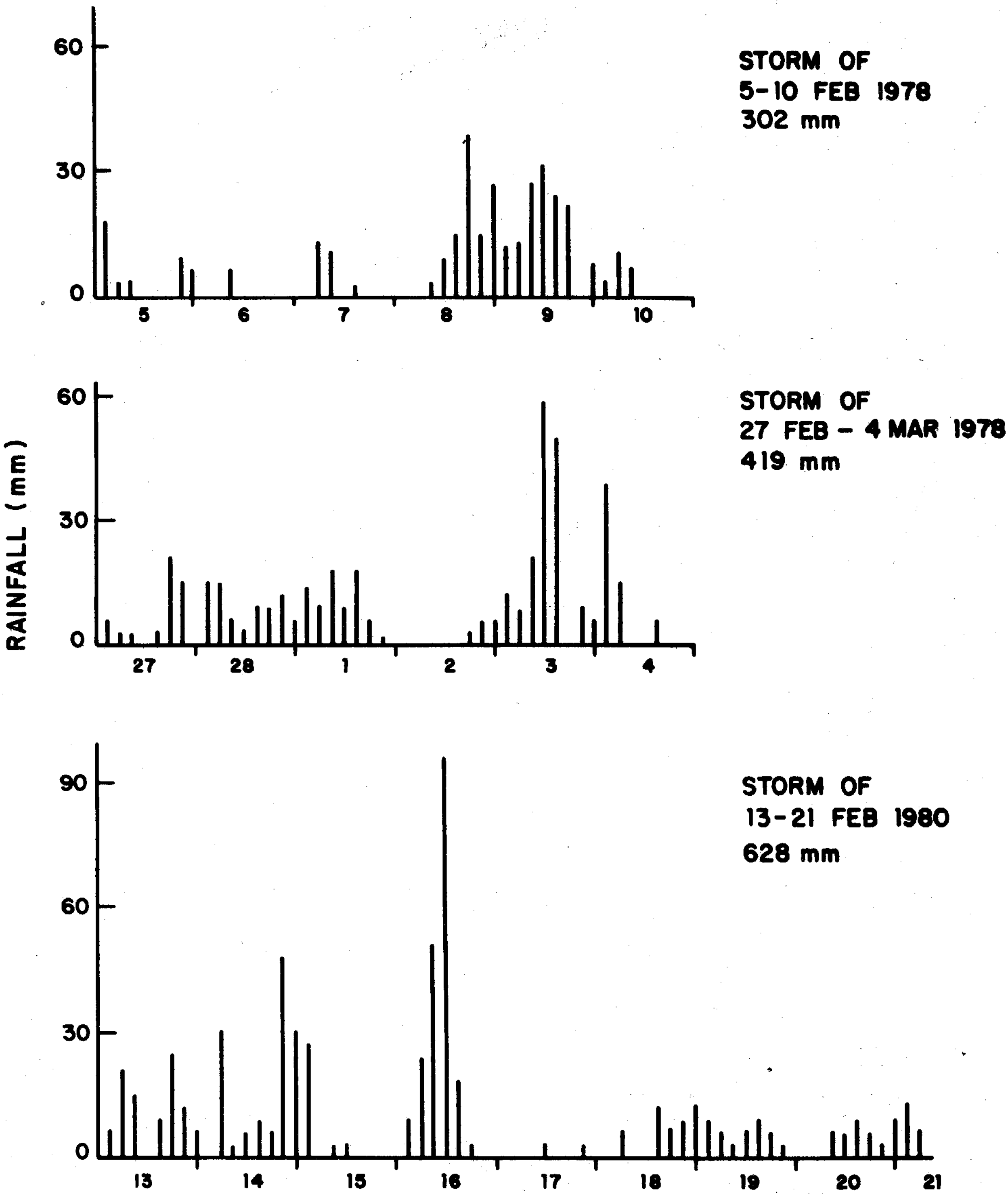


FIGURE 4 Hyetographs of the major storms of 1978 and 1980. (Bars represent rainfall for three-hour periods ending at times indicated; time ticks are at midnight.) Data are from rain gage at Tanbark Flat.

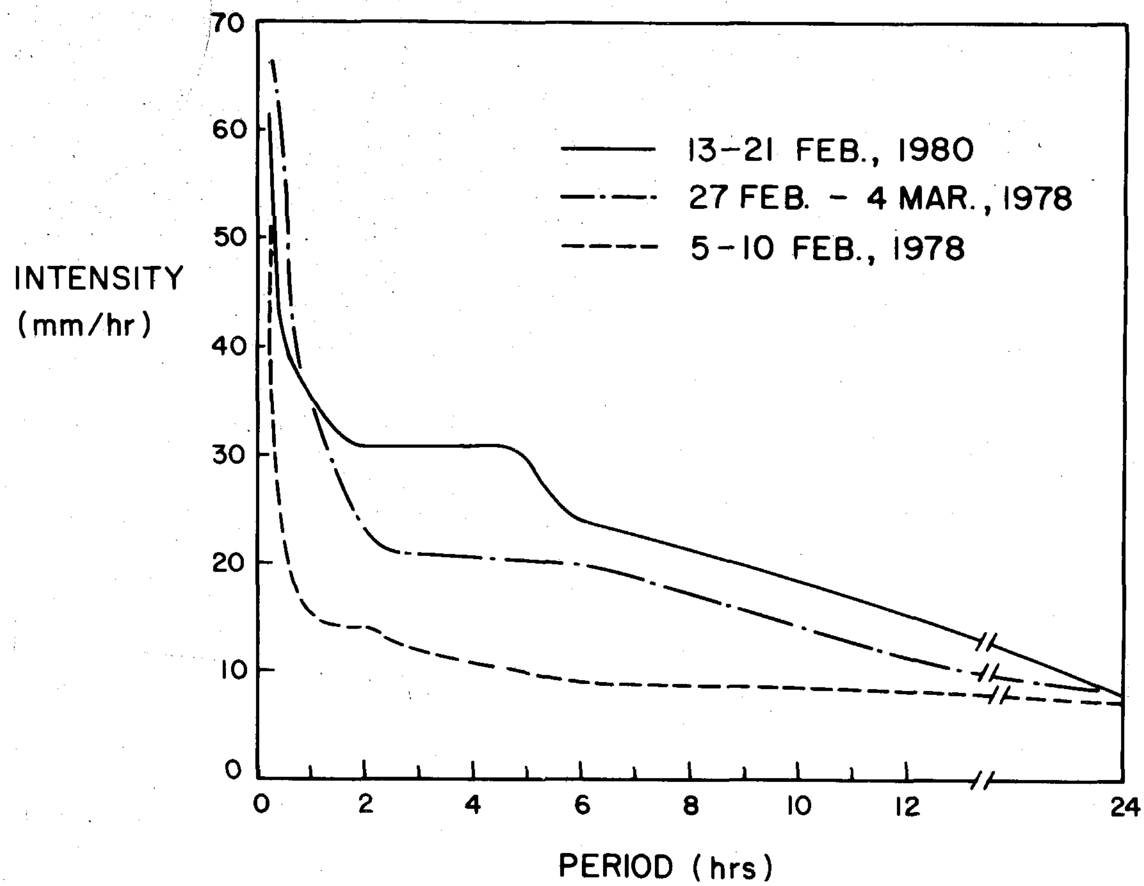


FIGURE 5 Peak rainfall intensities recorded at Tanbark Flat during selected time periods for the major storms of 1980 and 1978.

the fact that the 1980 storm was over twice as big and exhibited much higher rainfall intensities. This is especially true in Bell 2. The Bell 3 figures show that peak runoff for the storm of February 27-March 4, 1978, was much lower, less than 60 percent that of the other two storms.

Second, debris production per centimeter of rainfall was substantially higher in 1978 than in 1980. Again, the data for Bell 2 are particularly striking. When the sizes of the two storms are considered, sediment production (cubic meters of sediment per centimeter of rainfall) by the storm of February 5-10, 1978, is about three times as great as that in 1980. The difference found in Bell 3, even though both 1978 storms are combined, is also impressive. If the observers' estimates are accurate, debris production by the earlier of the 1978 storms alone would be almost 2-1/2 times the 1980 production. Two views of the Bell 3 debris basin in Figure 6 show the difference between 1978 and 1980 in debris production from that watershed. The effect of the 1975 fire in Bell 2 appeared to be negligible. No appreciable debris production was noted in the erosion troughs after either 1978 storm (see Figure 7).

TABLE 2 Peak Runoff and Sediment Delivery from the Bell Watersheds During the Major Storms of 1978 and 1980

Storm	Total Rainfall (cm)	Peak Runoff (liters/s)	Sediment Delivery[a] Total (cu m)	(1)	(2)	(3)
Bell 2 (40 ha)						
Feb. 13-21, 1980	62.8	1,399	351	8.78	5.59	0.14
Feb. 5-10, 1978	30.2	1,382	490	12.25	16.23	0.41
Bell 3 (25 ha)						
Feb. 13-21, 1980	62.8	599	268	10.72	4.27	0.17
Feb. 5-10, 1978	30.2	335				
			477	19.08	6.62	0.26
Feb. 27-Mar. 4, 1978	41.9	564				

[a](1) Cubic meters of sediment per hectare; (2) cubic meters of sediment per centimeter of rainfall; (3) cubic meters of sediment per hectare per centimeter of rainfall.

DISCUSSION

While the storm of February 13-21, 1980, was the most severe of the three storms considered in this report, it moved considerably less sediment. This is particularly true in Bell 2, where a 302-mm storm moved 1.4 times as much sediment as a storm over twice its size and intensity. Differences in the timing of rainfall are not great enough to account for this, and, since Bell 3 showed a similar pattern of sediment production, residual fire effects cannot fully account for it either.

The most plausible explanation for these rather marked differences in sediment delivery is that there was considerably more sediment available for movement in 1978 than there was in 1980. Anderson et al. (1959) found that watersheds in the San Gabriel front exhibited recognizable patterns of channel filling during the dry season and channel scour during storms. Krammes (1965) noted that channel filling occurred throughout the year but that channel scour occurred only during the larger storms. Because the degree of scour largely depends on the size of channel flows, it would seem that a long-term cycle of filling and scouring also exists with respect to the occurrence of major storms. Sediment that is protected from scour during normal flows by armoring of the bed may become vulnerable to scour by high flows that can move larger particles and, thus, the smaller particles shielded by them.

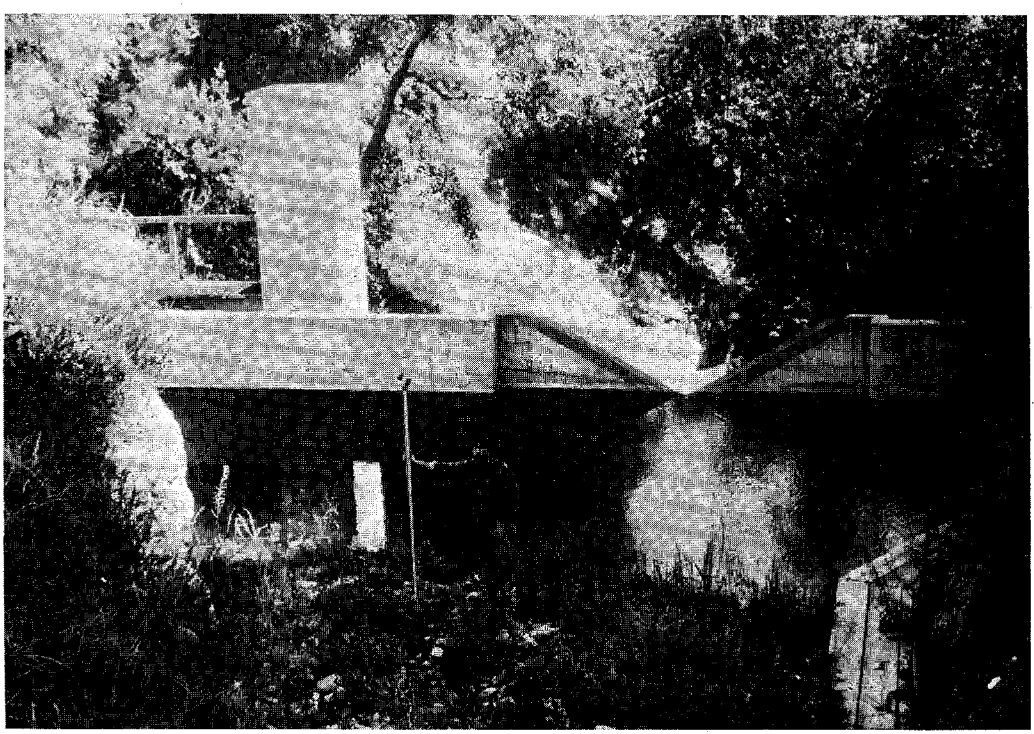


FIGURE 6 Views of Bell 3 debris basin in 1978 (top) and 1980 (bottom). Bell 3 had about one third of its capacity remaining after the 1980 storm but was completely filled by the 1978 storms.

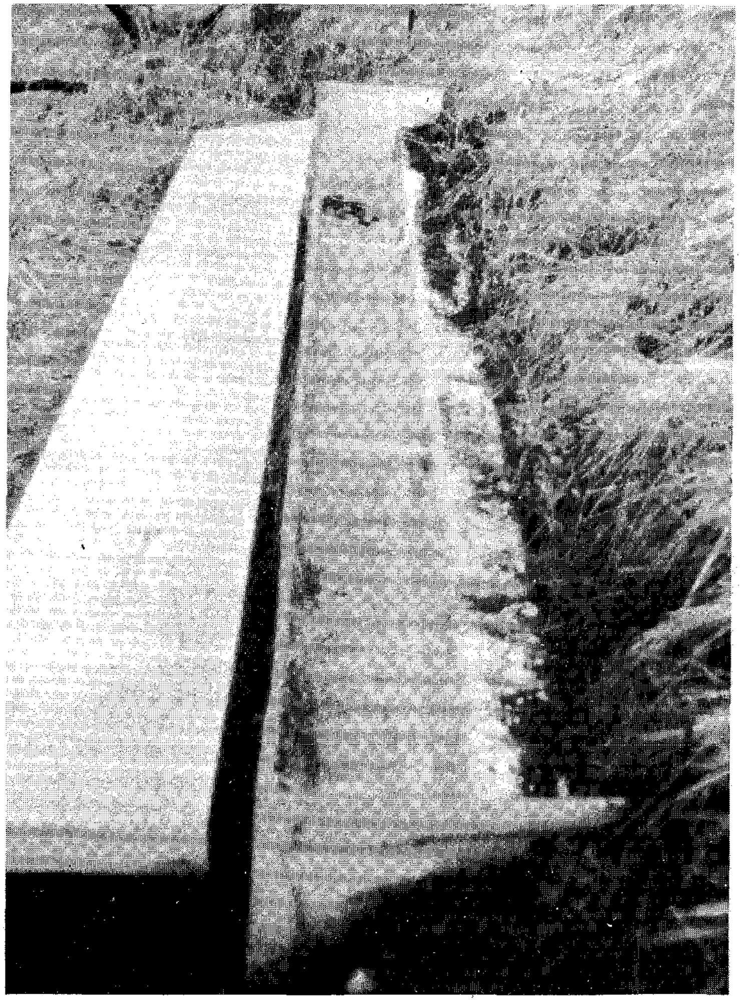


FIGURE 7 Debris trough on hillslope above the Bell 2 channel after the storm of February 5-10, 1978. This 3-m-long trough was installed after a fire in 1975 to monitor postfire erosion. No significant amounts of sediment were caught in this trough after January 1978. The trough was removed in 1979. Note the heavy grass and forb cover, which is typical of the vegetation on Bell 2. Since it was converted from chaparral to grass in 1961, annual sediment production from this watershed has been about 2-1/4 times as great as before conversion.

If this conclusion is correct, sediment delivery for a given storm should be related to the period of elapsed time between it and the last previous storm of comparable or greater size. It is interesting to note that sediment delivery for the four major storms of the last 50 years generally fits this pattern. The storms of 1969 yielded the most sediment, after an interval of 31 years between major storms (1938-69). The 1938 storm was second, after an interval of 22 years (1916-38). The year 1978 was third, with a nine-year interval (1969-78), and 1980 had only a two-year period for fresh sediment to collect in the channels.

FIRE EFFECTS

At the beginning of the study a synoptic assessment was made of sediment movement along the entire San Gabriel front, and the effect of recent fires in the area between Arroyo Seco and the San Gabriel River was particularly striking. Between 1977 and 1980 two major fires, the Mountain Trail fire of October 1978 and the Pinecrest fire of September 1979, burned a large part of this area. Records from 14 debris basins maintained by the Los Angeles County Flood Control District were used to compare debris production from this area in 1978 and 1980.

The data from these basins are summarized in Table 3. Sediment production from the watersheds in this area fell into three distinct groups, one of recently burned watersheds and two of unburned watersheds, that corresponded to rather well-defined sections of the mountain front. Group 1 consisted of unburned watersheds between Arroyo Seco and Eaton Wash (see Figure 1). Sediment production from this group was about three times as great in 1978 as in 1980. Group 2 included the watersheds between Eaton Wash and the San Gabriel River. Sediment production from them was three times as great in 1980 as in 1978, just the reverse of group 1. Group 3 consisted of watersheds in both areas that had been totally burned in either 1978 or 1979. These burned watersheds produced 10 times as much sediment in 1980 as in 1978. They produced 5 times as much sediment as those in group 2 and 17 times as much sediment as those in group 1.

Of the burned watersheds, Rubio and Las Flores are located in the group 1 area and were burned in September 1979. The other four were located in the group 2 area, were burned in October 1978, and therefore had already been recovering for one growing season before the 1980 storm. Nevertheless, there was no appreciable difference in the debris production among the six burned watersheds.

The relationship between fires and flooding in southern California has been recognized for over 50 years (Wells, 1981), but very little work has been done toward understanding the processes that cause this postfire flooding. The first comprehensive study of the problem was done by Rowe et al. (1954) and, for many areas, provides the only information on the problem available today. Certain aspects of the problem, such as the occurrence of fire-induced water-repellent soils, have been well studied (DeBano, 1981; DeBano et al., 1979). Preliminary studies of other aspects of the problem have also been made (Krammes, 1960; San Dimas Staff, 1954; Davis, 1977; Wells, 1981; Wells and Brown, 1981), but detailed studies of the processes involved have not been done.

SUMMARY AND CONCLUSIONS

In this report we have examined the major storms of 1978 and 1980 and their effect on sediment movement in two small catchments in the eastern San Gabriel Mountains. We then looked, briefly, at the effect of recent fires on sediment movement by these same storms. From this brief study we can conclude that large sediment movements are not necessarily related to storm size but

TABLE 3 Sediment Production from Burned and Unburned Watersheds in the San Gabriel Front Between Arroyo Seco and the San Gabriel River

Watershed Name	Area (ha)	Sediment Production (cu m/ha) 1978	1980
Group 1: Unburned watersheds located between Arroyo Seco and Eaton Wash			
Fern	78	72.0	21.1
Fair Oaks	54	36.5	4.6
West Ravine	65	41.9	17.9
Lincoln	130	33.2	20.9
Mean sediment production (cu m/ha)		45.9	16.1
Standard deviation of sample (cu m/ha)		15.4	6.8
Standard deviation as % of the mean		33.5	42.1
Group 2: Unburned watersheds located between Eaton Wash and the San Gabriel River			
Ruby	73	2.4	34.9
Bradbury	176	35.8	75.1
Spinks	114	23.4	63.2
Maddok	65	5.1	64.4
Mean sediment production (cu m/ha)		16.7	59.4
Standard deviation of sample (cu m/ha)		13.7	14.9
Standard deviation as % of the mean		82.2	25.1
Group 3: Burned watersheds located in areas covered by both groups 1 and 2			
Carter (burned 1978)	31	10.2	262.4
Auburn (burned 1978)	49	15.0	306.3
Lannan (burned 1978)	65	37.7	106.1
Bailey (burned 1978)	155	31.8	447.9
Rubio (burned 1979)	326	15.9	311.7
Las Flores (burned 1979)	116	51.9	233.5
Mean sediment production (cu m/ha)		27.1	278.0
Standard deviation of sample (cu m/ha)		14.8	102.1
Standard deviation as % of the mean		54.5	36.7

seem to be related to several factors. Two of these factors would be the amount of sediment available for movement in stream channels and the immediate fire history of the area. It is probable that the former is strongly influenced by the latter.

We have seen an example of a relatively small (though still quite severe) storm moving more sediment than a much larger storm and an example of the tremendous effect that brushfires in southern California can have on sediment production. Although possible explanations for these phenomena can be offered, direct proof is lacking and it is clear that we have only a superficial understanding of the sedimentation processes that occur on natural watersheds. In particular, we do not really understand the exact role of the various hillslope processes in supplying sediment to natural channels, nor do we understand how their dominance may change over time. It appears that a large percentage of sediment is supplied by a very small percentage of the total area, but sediment budget studies to confirm or refute this are not yet available. The effects of fire are only beginning to be understood, but it appears that a unique set of erosional processes dominate the postfire environment; more work is needed to understand them fully. Mudflows are common in California's mountainous areas, but relatively little is known about their causes or mechanics of movement.

Erosion and sediment movement in the arid West are driven by extreme events like the storms of 1978 and 1980. The more frequent, lower-intensity events that tend to dominate in humid regions have a much lesser effect here. Our methods for dealing with such events are not perfect, and some practices, such as seeding steep slopes to grass, may not always be appropriate. A better understanding of the underlying processes will help us deal more effectively with these events and their consequences.

REFERENCES

Anderson, H. W., Coleman, G. B., and Zinke, P. J. (1959) Summer Slides and Winter Scour, Technical Paper 36, Pacific Southwest Forest and Range Experiment Station, U.S. Department of Agriculture, Forest Service, Berkeley.

Davis, J. D. (1977) Southern California reservoir sedimentation, American Society of Civil Engineers, fall meeting and exhibits, San Francisco.

DeBano, L. F. (1981) Water Repellent Soils: A State of the Art, General Technical Report PSW-46, Pacific Southwest Forest and Range Experiment Station, U.S. Department of Agriculture, Forest Service, Berkeley.

DeBano, L. F., Rice, R. M., and Conrad, C. E. (1979) Soil Heating in Chaparral Fires: Effects on Soil Properties, Plant Nutrients, Erosion and Runoff, Research Paper PSW-145, Pacific Southwest Forest and Range Experiment Station, U.S. Department of Agriculture, Forest Service, Berkeley.

Krammes, J. S. (1960) Erosion from Mountain Side Slopes after Fire in Southern California, Research Note No. 171, Pacific Southwest Forest and Range

Experiment Station, U.S. Department of Agriculture, Forest Service, Berkeley.

Krammes, J. S. (1965) Seasonal debris movement from steep mountain side slopes in southern California, Proceedings of the Federal Interagency Sedimentation Conference, 1963, Miscellaneous Publication 970, U.S. Department of Agriculture, Washington, D.C., pp. 85-89.

San Dimas Staff (1954) Fire-Flood Sequences in Southern California, Pacific Southwest Forest and Range Experiment Station, U.S. Department of Agriculture, Forest Service, Berkeley.

Rowe, P. B., Countryman, C. M., and Storey, H. C. (1954) Hydrologic analysis used to determine the effects of fire on peak discharges and erosion rates, California Forest and Range Experiment Station, U.S. Department of Agriculture, Forest Service, Berkeley.

Wells, W. G. II (1981) Some effects of brushfires on erosion processes in coastal southern California, Erosion and Sedimentation in Pacific Rim Steeplands, I.A.H.S. Publication No. 132, Christchurch, New Zealand.

Wells, W. G. II, and Brown, W. M. III (1981) Effects of fire on sedimentation processes, Sediment Management for Southern California Mountains, Coastal Plains and Shoreline, EQL Report No. 17-D, Environmental Quality Laboratory, California Institute of Technology, Pasadena.

RARE AND UNUSUAL POSTFIRE FLOOD EVENTS EXPERIENCED IN LOS ANGELES COUNTY DURING 1978 and 1980

by J. Daniel Davis

Several areas in Los Angeles County experienced extreme floods during the storms of 1978 and 1980. In most cases the most important factor determining the severity of the floods was the fire history of the area. Much of the debris deposited in debris basins can be directly attributed to burned watersheds.

This paper examines flood events in four separate locations--Hidden Springs, Zachau Canyon, Shields Canyon, and Rubio Canyon. In all four cases the flood waters came largely from areas that had been recently burned. These events have an estimated return period of 200 to 1,000 years.

Los Angeles County comprises a land area of approximately 4,000 square miles (10,000 sq km). Forty-seven percent of the area is mountainous, the remainder being alluvial valleys and coastal plains. Storms and subsequent flooding usually occur during the winter storm season. Periodically throughout the history of Los Angeles County, major flooding has occurred due to heavy winter rains.

The major factors in the degree of damage incurred in 1978 and 1980 were the Mill fire of November 1975, the Village fire of November 1975, the Middle Fork fire of July 1977, and the Pinecrest fire of September 1979. These fires consumed 208 square miles (539 sq km) of watershed above urban areas or above dams and debris basins owned and operated by the Los Angeles County Flood Control District. Table 1 summarizes the extent of these fires and Figure 1 shows their locations.

The primary impact of the storms of 1978 and 1980 was in the foothills and mountains. Due to the degree of drainage improvements in Los Angeles County, and to the fact that storm intensities were generally less than the design level of the systems, flooding in urban areas was held to a low level. Table 2 summarizes the damages to public facilities for each storm year. The

---

J. Daniel Davis is Head of the Operations Section, Hydraulic Division, with the Los Angeles County Flood Control District in Los Angeles, California.

TABLE 1 Fires Contributing to Storm Damages of 1978 and 1980

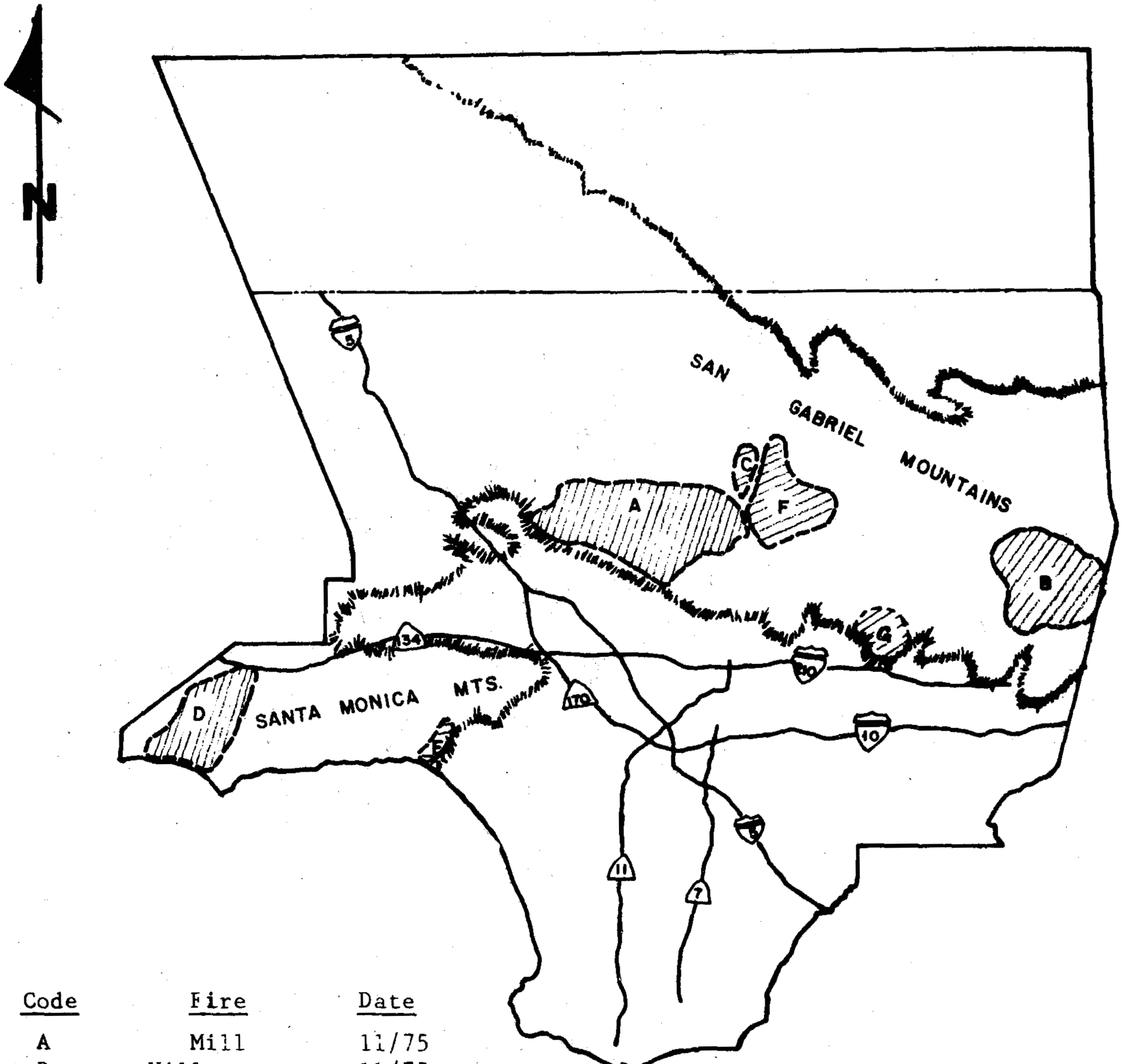
Name	Date of Burn	Area Burned: Square Miles	Area Burned: Square Kilometers
Mill fire	November 1975	70.3	182
Village fire	November 1975	31.2	81
Middle Fork fire	July 1977	5.9	15
Kanan (Malibu) fire	October 1978	39.1	101
Mandeville fire	October 1978	8.4	22
Sage fire	September 1979	45.3	117
Pinecrest fire	September 1979	8.3	21
Total		208.5	539

majority of the damage in foothill areas is attributable to cleanout costs incurred in removing debris from debris basins and emergency structures built to control runoff from burned watersheds.

The storm of February 9-10, 1978, resulted in the major damages of the 1978 storm season. Rainfall records indicate that an intense cell of rain crossed a strip of the county. It started at Santa Monica and moved northeast across San Fernando Valley, the western half of the Santa Monica Mountains, through the Sunland-Tujunga area, and into the Big Tujunga watershed. The cell dissipated high in the San Gabriel Mountains near the vicinity of the Hidden Springs disaster. Rainfall records and other evidence show that the cell still contained a significant amount of rain as it passed the Middle Fork Canyon above Hidden Springs. Figure 2 shows the path and recurrence interval associated with this cell and locates the areas of major problems discussed in this paper.

TABLE 2 Damage to Public Facilities During 1978 and 1980

Storm Year	Damage in Foothills and Mountains (dollars)	Damage in Urban Areas (dollars)
1978	31,500,000	3,000,000
1980	46,000,000	?



Code	Fire	Date
A	Mill	11/75
B	Village	11/75
C	Middle	7/77
D	Kanan (Malibu)	10/78
E	Manderville	10/78
F	Sage	9/79
G	Pinecrest	9/79

FIGURE 1 Major fires during the period 1975-79.

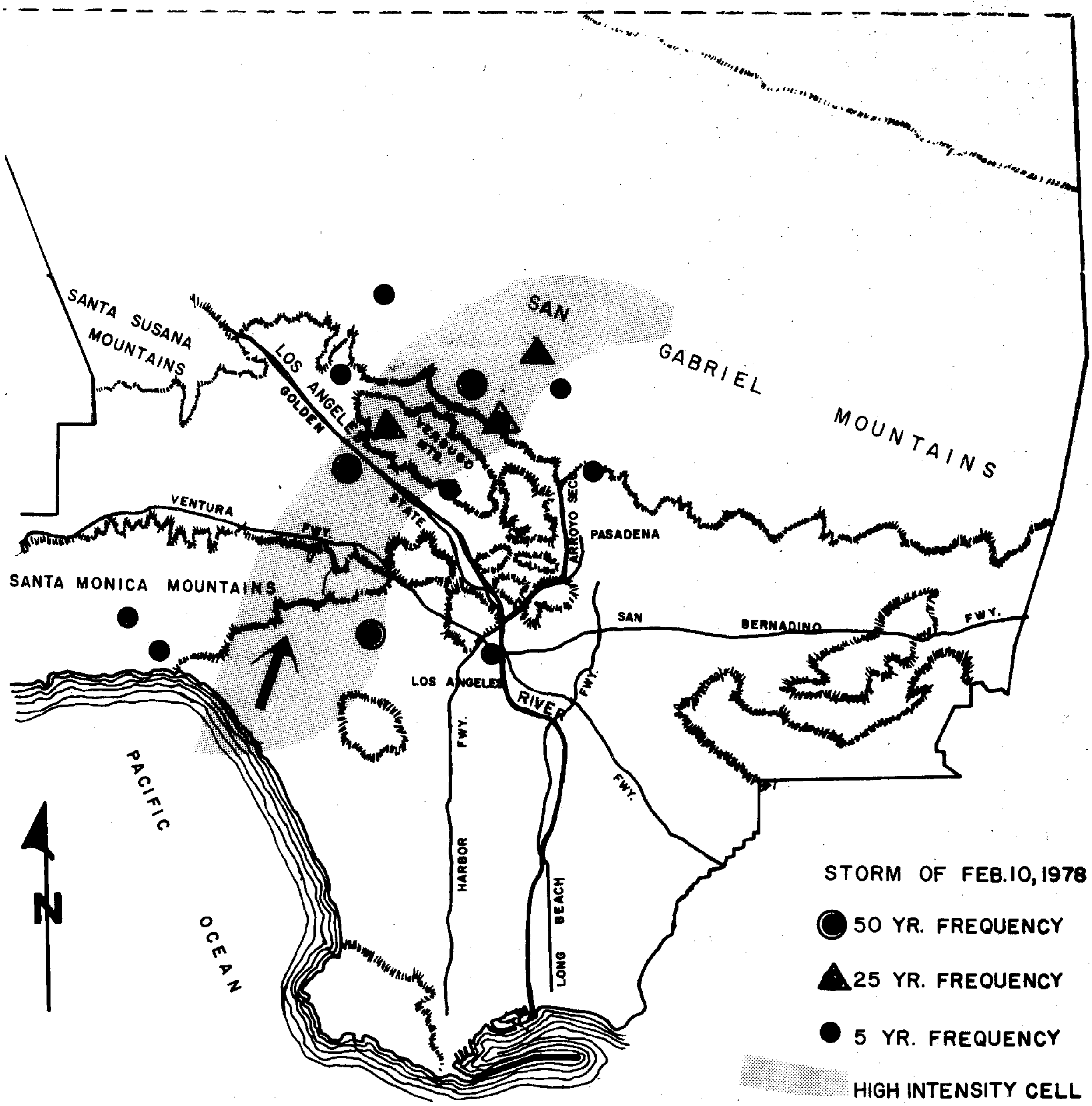


FIGURE 2 Path of the February 10, 1978, storm.

DEBRIS PRODUCTION

The total debris production for the 1978 and 1980 storm seasons is given in Table 3. Table 4 gives the production amounts for individual debris basins.

Watershed burns were a major cause of debris production during the storm years. Of the total of 2,250,000 cu yd (1,720,000 cu m) of debris deposited

TABLE 3 Debris Production During the 1978 and 1980 Storm Seasons

Season	Debris Production Debris Basins (cu yd)[a]	Dams (cu yd)[a]	Total (cu yd)[a]
1978	1,200,000 (920,000)	8,800,000 (6,700,000)	10,000,000 (7,600,000)
1980	1,050,000 (800,000)	5,400,000 (4,100,000)	6,450,000 (4,900,000)

[a]Figures in parentheses give the production amounts in cubic meters.

in district debris basins in 1978 and 1980, 900,000 cu yd (688,000 cu m) can be directly attributed to watershed burns.

The impact of a burn can best be seen from Figure 3. The graph shows the unit rate of production for the debris basins as a function of watershed area. It is obvious from the plot that almost every large event was due to a burned watershed. From Figure 3 it is difficult to generalize any relationship between debris production and watershed area. It is interesting to note that the median burned-area production (26,240 cu m/sq km) exceeds the median unburned-area production (4,220 cu m/sq km) by a factor of 6.2.

HIDDEN SPRINGS

Deep in the San Gabriel Mountains, above the Sunland-Tujunga area, is a small resort area called Hidden Springs. The area is used as a camping and recreational escape from the city. A couple of small restaurants, a gas station, a swimming pool, campsites, and other improvements associated with mountainous recreation areas dot the canyon floor.

In July 1977 a fire occurred above Hidden Springs, resulting in the complete burn of Middle Fork Canyon, a 2,440-acre (9.87-sq-km) watershed. On February 8, 9, and 10, 1978, rainfall in the Big Tujunga watershed resulted in torrential runoff out of Middle Fork Canyon. The flow rate at the mouth of the canyon was estimated to be 9,000 cu ft/s (255 cu m/s). The flow was described by an observer as a wall of water 30 ft high (9 m). Actual cross sections indicated a maximum depth of flow of about 15 ft (4.5 m).

Middle Fork Canyon is 25 percent of the total watershed area above Hidden Springs, which amounts to 10,000 acres (41.0 sq km). However, because of the watershed burn, estimates indicate that 97 percent of the peak flow that inundated Hidden Springs was due to the Middle Fork fire.

The damage toll in Hidden Springs was nearly complete for all facilities on or near the bottom of the canyon. Several people camping in the area were never found and were presumed swept away by the flood.

TABLE 4 Debris Production for Individual Debris Basins During the 1978 and 1980 Storm Seasons

No.	Debris Basin	Watershed Area		Debris Production 1978			Debris Production 1980			Burn History	
		Sq Mile	Sq Km	Cu Yd/ Sq Mile	Cu M/ Sq Km	In Terms of Mean	Cu Yd/ Sq Mile	Cu M/ Sq Km	In Terms of Mean	Year	Percent
1	Aliso	2.77	7.17	9,700	2,860	3.25	6,600	1,950	2.21	1970	66
2	Auburn	0.19	0.49	16,100	4,750	3.82	88,900	26,240	21.08	1977	100
3	Bailey	0.60	1.55	22,500	6,640	6.28	145,200	42,860	40.55	1977	100
4	Big Dalton	2.62	6.79	31,100	9,180	5.18	38,800	11,450	6.47	1960	100
5	Blanchard	0.50	1.29	73,200	21,610	9.15	14,500	4,280	1.83	1975	88
6	Blue Gum	0.19	0.49	100,600	29,700	12.58	13,400	3,960	1.68	1975	100
7	Bradbury	0.68	1.76	12,100	3,570	1.37	25,500	7,530	2.89	1958	100
8	Brand	1.03	2.67	51,600	15,230	15.75	22,000	6,490	6.71	1927	66
9	Carter	0.12	0.31	200	60	0.12	88,900	26,240	53.88	1977	100
10	Childs	0.31	0.80	13,900	4,100	2.39	7,700	2,270	13.26	1927	100
11	Cooks	0.58	1.50	102,200	30,170	23.35	22,000	6,490	5.03	1975	100
12	Deer	0.59	1.53	37,600	11,100	9.27	11,200	3,310	2.76	1964	100
13	Dunsmuir	0.84	2.18	98,700	29,140	27.33	22,800	6,730	6.31	1975	100
14	Eagle	0.48	1.24	60,300	17,800	6.54	10,300	3,040	1.12	1975	100
15	Englewild	0.40	1.04	2,800	830	0.28	30,900	9,120	3.06	1968	100
16	Fair Oaks	0.21	0.54	12,400	3,660	1.66	150	44	0.02	1935	100
17	Fern	0.30	0.78	14,600	4,310	1.91	7,100	2,100	0.93	1962	48
18	Gould	0.47	1.22	8,400	2,480	1.39	7,100	2,100	1.18	1959	82
19	Halls	1.06	2.75	43,600	12,870	5.82	18,400	5,430	2.45	1933	85
20	Harrow	0.43	1.11	5,100	1,510	0.63	7,500	2,210	0.92	1968	100
21	Hay	0.20	0.52	21,700	6,410	6.73	0	0	0	1959	100
22	Hillcrest	0.35	0.91	16,000	4,720	4.67	3,800	1,120	1.11	1964	100
23	Hog	0.30	0.78	12,800	3,780	10.10	8,600	2,540	6.79	1962	100
24	Hook East	0.18	0.47	11,900	3,510	1.49	13,200	3,900	1.65	1968	100
25	Hook West	0.17	0.44	10,800	3,190	1.35	21,200	6,260	2.65	1968	98
26	Kinneloa	0.20	0.52	16,500	4,870	1.94	8,900	2,630	1.05	No Recorded Burn	
27	Kinneloa West	0.16	0.41	22,300	6,580	2.62	20,300	5,990	2.39	"	

28	Lannan	0.25	0.65	12,800	3,780	1.71	35,900	10,600	4.79	1969	80
29	Las Flores	0.45	1.17	17,400	5,140	6.05	74,400	21,960	25.89	1979	95
30	La Tuna	5.34	13.83	32,200	9,510	11.42	14,300	4,220	5.07	1955	61
31	Limekiln	3.69	9.56	10,200	3,010	2.89	5,800	1,710	1.64	1970	95
32	Lincoln	0.50	1.29	11,000	3,250	3.23	5,700	1,680	1.67	1935	98
33	Little Dalton	3.31	8.57	22,400	6,610	3.73	30,200	8,920	5.03	1960	89
34	Maddock	0.25	0.65	1,700	500	0.38	21,800	6,440	4.82	1958	95
35	May No. 1	0.70	1.81	10,400	3,070	2.04	3,200	940	0.63	1966	100
36	May No. 2	0.09	0.23	6,600	1,950	1.30	6,200	1,830	1.22	1966	100
37	Morgan	0.60	1.55	7,100	2,100	3.47	9,700	290	4.75	No Recorded Burn	
38	Nichols	0.35	0.91	6,700	1,980	1.36	29,200	8,620	5.93	No Recorded Burn	
39	Pickens	1.50	3.88	89,900	26,540	18.09	15,800	4,660	3.18	1975	86
40	Rowley	0.27	0.70	174,800	5,160	32.00	27,700	8,180	5.07	1975	58
41	Rubio	1.26	3.26	5,400	1,590	1.42	84,557	24,960	22.20	1979	100
42	Ruby Lower	0.28	0.73	180	53	0.07	11,800	3,483	4.64	1953	88
43	Santa Anita	1.70	4.40	23,800	7,030	3.97	12,400	3,660	2.07	1969	23
44	Sawpit	2.78	7.20	7,900	2,330	1.32	13,200	3,990	2.20	1958	60
45	Schoolhouse	0.28	0.73	5,700	1,680	0.81	0	0	0	1962	100
46	Shields	0.23	0.60	117,200	34,600	15.35	49,400	14,580	6.47	1975	87
47	Sierra Madre Villa	1.46	3.78	8,000	2,360	1.33	64,200	18,950	10.70	1978	53
48	Snover	0.23	0.60	7,300	2,150	0.97	18,000	5,310	2.40	1933	97
49	Spinks	0.44	1.14	7,900	2,330	1.64	21,300	6,290	4.42	1953	17
50	Stetson	0.29	0.75	5,300	1,560	5.70	4,800	1,420	5.16	1962	100
51	Sullivan	2.38	6.16	8,900	2,630	5.20	14,800	4,370	8.64	1979	45
52	Sunset Upper	0.44	1.14	43,000	12,690	9.38	31,600	9,330	6.89	1964	99
53	Turnbull	0.99	2.56	2,800	830	4.55	4,300	1,270	6.99	1967	86
54	Ward	0.12	0.31	148,100	43,720	37.60	38,900	11,480	9.88	1975	100
55	West Ravine	0.25	0.65	8,300	2,450	1.18	6,100	1,800	0.87	1935	100
56	Wildwood	0.65	1.68	23,000	6,790	3.44	12,700	3,750	1.90	1957	76
57	Wilson	2.58	6.68	5,500	1,620	1.34	2,058	610	0.50	1966	35
58	Winery	0.18	0.47	2,700	800	0.36	20	6	0	1962	36
59	Zachau	0.35	0.91	141,800	41,860	18.91	26,400	7,790	3.52	1975	98

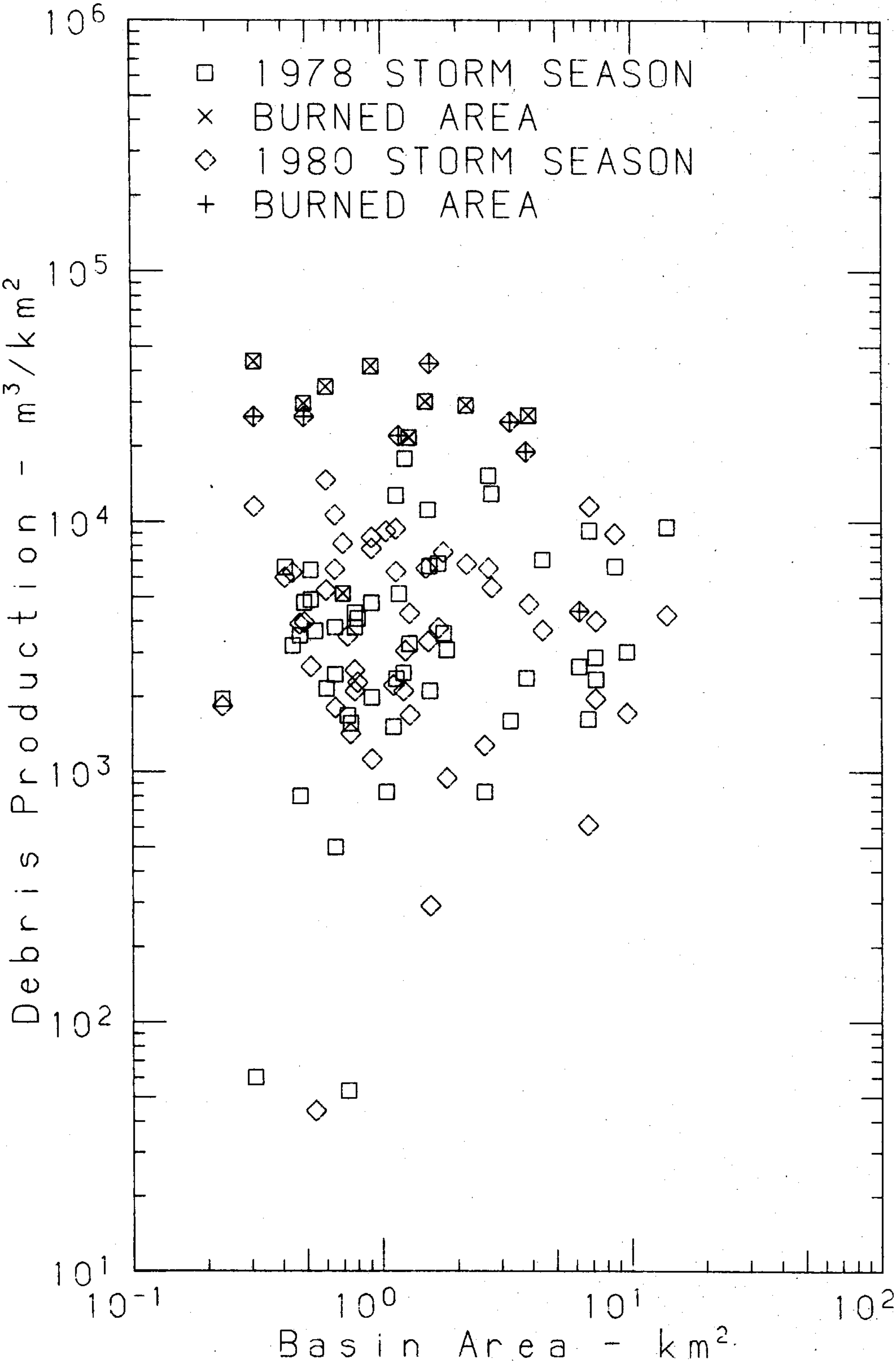


FIGURE 3 Debris production rates during 1978 and 1980.

The floodwave that swept through Hidden Springs was of the magnitude of the design flood. However, the potential power of the watershed was not tested. One wonders what event would have resulted if the storm had occurred in 1980, when the entire watershed above Hidden Springs was in a burned condition.

ZACHAU CANYON

Zachau Canyon is a 0.58-square-mile (1.50-sq-km) watershed located above Sunland. The watershed was completely burned by the Mill fire of November 1975. The residential development at the mouth of the canyon and in the lower watershed is protected by several flood control improvements, including a debris basin and improved channel.

The Zachau debris basin was constructed in 1956 to keep debris-laden floodflow from entering into the rapidly developing floodplain below the canyon. The period from the construction of the basin until 1978 saw few notable events recorded at the basin. The 1969 storm produced erosion rates of 19,000 cu yd/square mile (8,600 cu m/sq km), an average debris runoff for that storm.

The 1978 storm cell described above hit Zachau Canyon and generated a runoff event that caused widespread damages in the watershed. The first improvement in the canyon is a road (Seven Hills Drive) crossing. The culvert under the road was plugged by a single boulder 6 to 7 ft (1.8 to 2.1 m) in diameter. Once the culvert plugged, flow inundated the roadway and proceeded downstream into the debris basin. The most dramatic effect of the storm was an 8-ft (2.4-m) diameter boulder that was deposited on Seven Hills Drive.

It is believed that sufficient debris had been deposited at the Zachau debris basin prior to the peak flow to decrease the capacity of the debris basin significantly. The floodwave went through the dam as if it were not there. The surge was large enough to crest the dam.

Immediately downstream of the debris basin the improved channel is open and recessed for the first reach. Flow through the basin left the channel but was contained by the channel way. Overflow estimates indicate that the actual flood magnitude was about twice the channel capacity. The channel was designed to contain the runoff from a storm having a 50-year intensity.

The major damages occurred when the open channel went underground. Although the underground drain functioned, it could not accept all the flow. Excess flow crossed the highway and entered the urban area. From where the flow left the improved channel to the next adequate collection point is a distance of about 7,000 ft (2,130 m). Between the two points the area is totally urbanized.

Below the open channel the flow split and took two paths. One route was to the west, down a street named Wentworth, and the other was to the northwest, down and across several streets. Approximately 50 residences along each path of flow experienced damages.

During the 1978 storm year 141,800 cu yd/square mile (41,860 cu m/sq km) of debris was deposited in the Zachau debris basin. Including debris that passed through the system, it is estimated that the erosion rate of the burned watershed during the 1978 season was 200,000 cu yd/square mile (59,000 cu m/sq km). This rate is equivalent to the loss of 2.3 in. (5.9 cm) of soil from the watershed.

SHIELDS CANYON

Shields Canyon is a 0.23-square-mile (0.60-sq-km) watershed above La Crescenta. The canyon is extremely steep, rising from an elevation of 2,600 ft (790 m) to 4,050 ft (1,230 m) in only 3,500 ft (1,070 m), a grade of 41 percent. Average sideslope grades are 100 percent.

Development in the 1960s encroached into this area, leapfrogging an existing debris basin and filling the mouth of the canyon. To protect the stabilization structures of the development, a small debris basin and an improved concrete channel were constructed to convey flows through the development to old debris basins.

The storm of February 9-10, 1978, resulted in a tremendous surge of material out of the canyon. The flow quickly filled the upper debris basin. At the peak of the storm, rocks and boulders were carried out of the canyon through the dam and into the improved drain. The peak flow rate exceeded spillway capacity, sending some flow down the access road onto Pine Cone Avenue below.

The major problem occurred when a 6-ft (1.8-m) boulder carried by the floodflow clogged the drain and forced all flow onto the street. Pine Cone Avenue runs north and south, generally paralleling the natural channel. The grade of the road is steep and the road has several curves. Houses have been constructed on both sides of the street.

On February 9, 1978, the flow that entered Pine Cone Avenue from the plugged drain could not be controlled by the streets. At the first curve a significant amount of flow left the street and went through a private residence, resulting in considerable damage to the property. As the flow worked its way down the street, homes on the curves or below ground level were inundated.

The most spectacular damage was to a residence at the end of Pine Cone Avenue immediately above the old debris basin. The house blocks the path of flow from entering the old debris basin. During the storm several automobiles were picked up by the large mudflow and swept down the street to the house. Figure 4 shows what the flood event did to this residence.

The debris production for Shields Canyon during the 1978 storm season was measured to be 117,000 cu yd/square mile (34,500 cu m/sq km). The total amount of debris produced by the 1978 storm in Shields Canyon was not sufficient to fill the lower debris basin. If the channel through the developed area had functioned, it is likely that little or no damage would have been incurred.

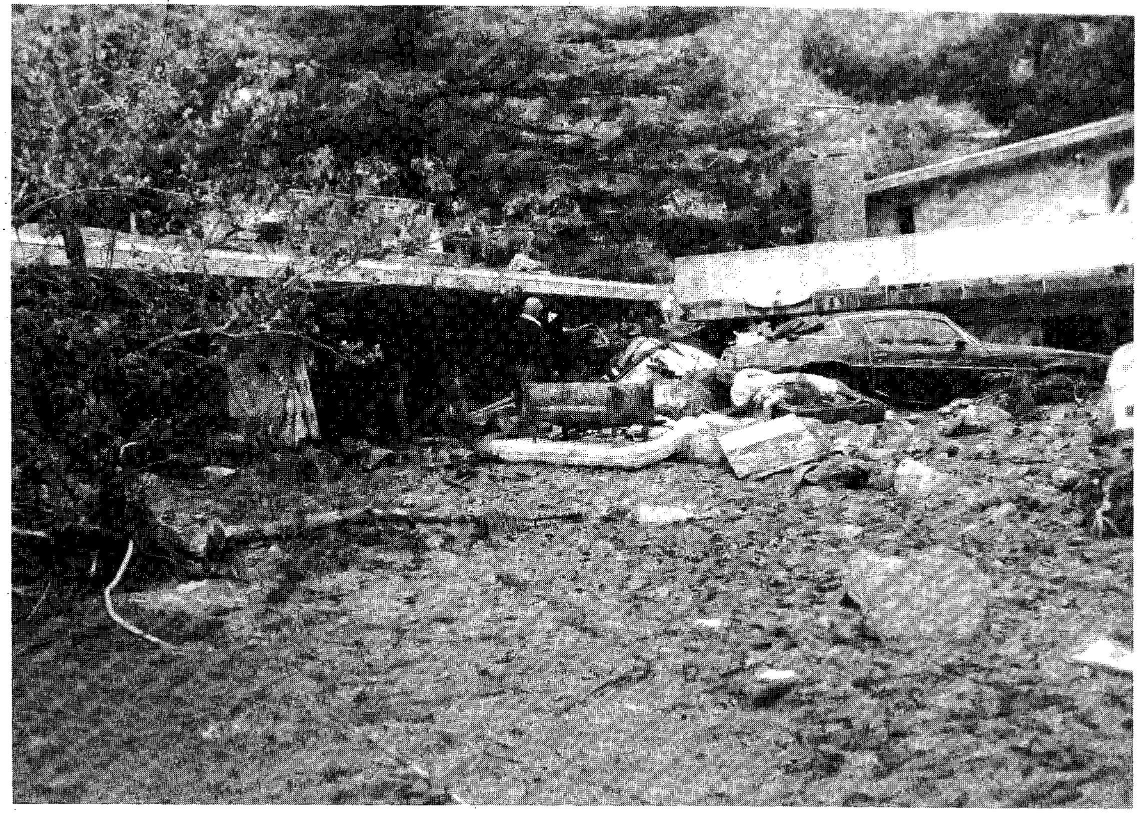


FIGURE 4 Storm damage at 2824 Markridge Road.

RUBIO CANYON

Rubio Canyon is a 1.26-square-mile (3.26-sq-km) watershed located above the Altadena area near Pasadena. The area is one of the older developed sections of the county. The first flood control improvements were constructed in the 1930s, and periodic improvements have been added as required. Figure 5 shows the configuration of the Rubio drainage system and the inundation area.

Las Flores Canyon, a tributary to Rubio Canyon, is controlled by a debris basin constructed in 1936. The Rubio debris basin was constructed in 1946. Rubio Wash, the channel below the basin, was constructed in the 1920s. However, due to the inadequacy of the wash, another channel called Rubio Diversion was completed by the Corps of Engineers in 1959. The Corps' construction involved intercepting flow below the Rubio debris basin and transporting it to Eaton Wash, a parallel drainage system to the east of Rubio.

The Rubio drainage system functioned with no problem until the 1980 storm season. During one of the first storms in 1980, sediment flow out of the Rubio debris basin deposited and clogged a section of the Rubio Diversion channel. Subsequent storms resulted in repeated deposition and filling of the channel due to flows out of both Rubio and Gooseberry canyons, until finally, on February 15, 1980, the channel clogged completely and flow was forced overland into the residential area.

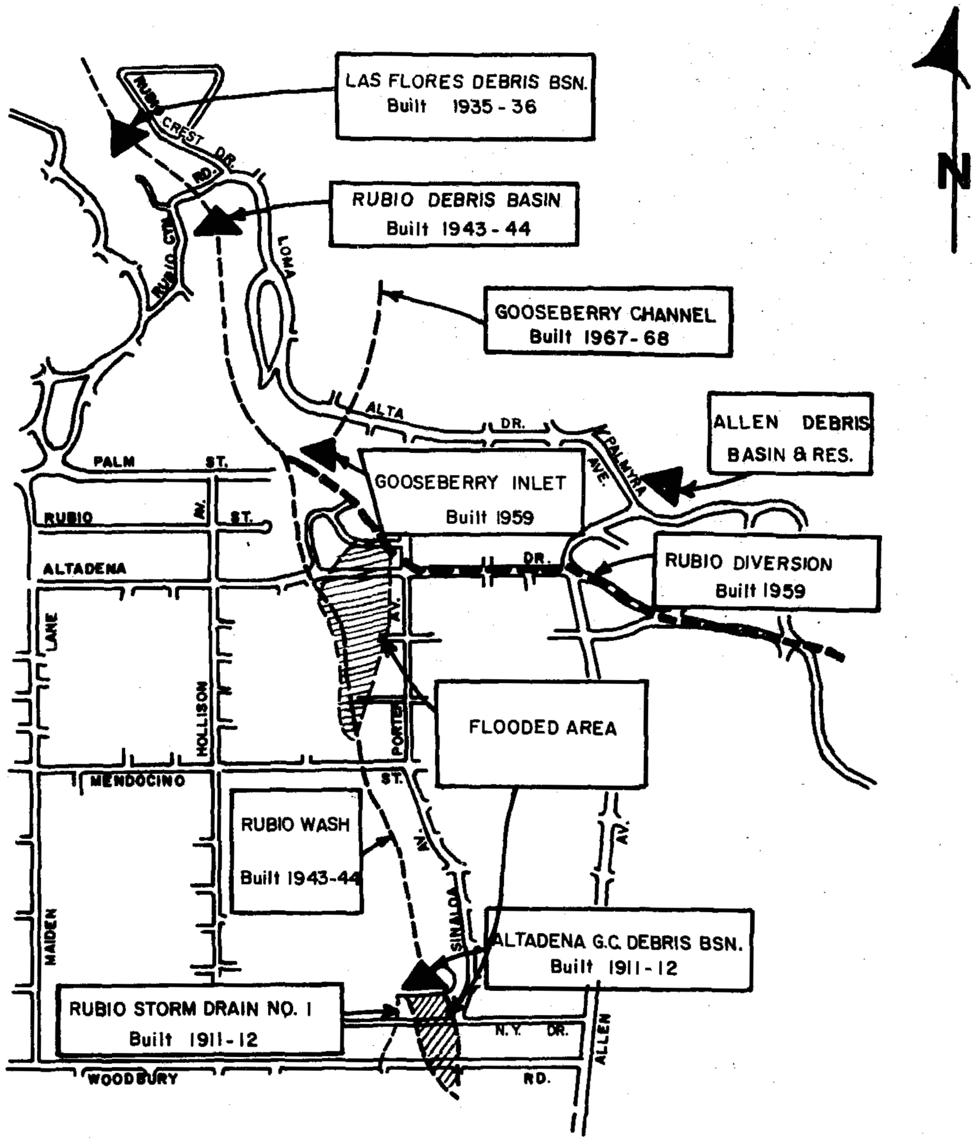


FIGURE 5 The Rubio Canyon drainage system and the areas inundated by the 1980 storms.

Two factors contributed to the debris deposition problem. First, in September 1979 the Rubio watershed burned as part of the Pinecrest fire. Second, the invert grades of the Rubio Diversion channel were significantly decreased from the natural gradient. Forcing the flow out of its natural waterway and across to the much larger Eaton Canyon channel required construction of several reaches of channel with a grade of one percent.

When the debris-laden runoff from the burned watershed met the reaches of decreased grades, the flow lost energy and deposition occurred until the channel plugged.

Damage from the overflow of Rubio Diversion was considerable. Approximately 30 homes were inundated and several automobiles were destroyed. Damages were recorded more than a mile downstream of the Rubio debris basin.

The debris production from Rubio Canyon of 85,000 cu yd/square mile (25,000 cu m/sq km) was less than the capacity of the debris basin. Much of the debris that contributed to the obstruction on February 15, 1980, came from Gooseberry Canyon. The small debris control structure in Gooseberry Canyon was filled by the storm runoff, sending a significant amount of material downstream to the Rubio Diversion channel.

SUMMARY

The storms of 1978 and 1980 resulted in major damage to the private and public sectors of Los Angeles County.

The high-intensity rainfall over the county and the previous watershed burns combined to create several extreme floods. Although mudflows in general are not rare and unusual in Los Angeles County, they are when an individual location is considered. Most of the events discussed in this paper are estimated to have approximately a 200- to 1,000-year level of recurrence.

THE SANTA BARBARA COUNTY FLOOD EXPERIENCE IN STORMS OF 1978 AND 1980

by James M. Stubchaer

The experience of the Santa Barbara County Flood Control and Water Conservation District during the 1978 and 1980 floods is discussed. Along the Santa Maria River, levees constructed by the Corps of Engineers in the early 1960s incurred partial failure. More complete failures, with the probability of costly local damage, were averted largely through the operation of levee patrol teams that identified weakened areas and implemented makeshift repairs. Experience from these flood years has helped identify needed improvements in this flood patrol system.

On the Santa Ynez River, a flood warning system developed locally was used to identify potential flood hazards based on a real-time analysis of rainfall data. This modeling effort proved accurate in predicting floodflows in 1978. Since 1978 this system has been improved upon and extended in geographic coverage.

Reclamation on the Sycamore Canyon watershed, which burned in July 1977, is also discussed. Poststorm observations indicate that these efforts were effective in reducing potentially severe flood damage on and below this watershed.

INTRODUCTION

This paper discusses the experience of the Santa Barbara County Flood Control and Water Conservation District with three facilities during the 1978 and 1980 floods.

SANTA MARIA RIVER LEVEES

The City of Santa Maria and the surrounding rich agricultural lands are located in the floodplain of the Santa Maria River (Figure 1). The river flows westerly to the Pacific Ocean in a channel varying in width from 1,400 to 2,200 ft. In 1960-62 the Corps of Engineers constructed the Santa Maria

---

James M. Stubchaer is Engineer-Manager with the Santa Barbara County Flood Control and Water Conservation District in Santa Barbara, California.

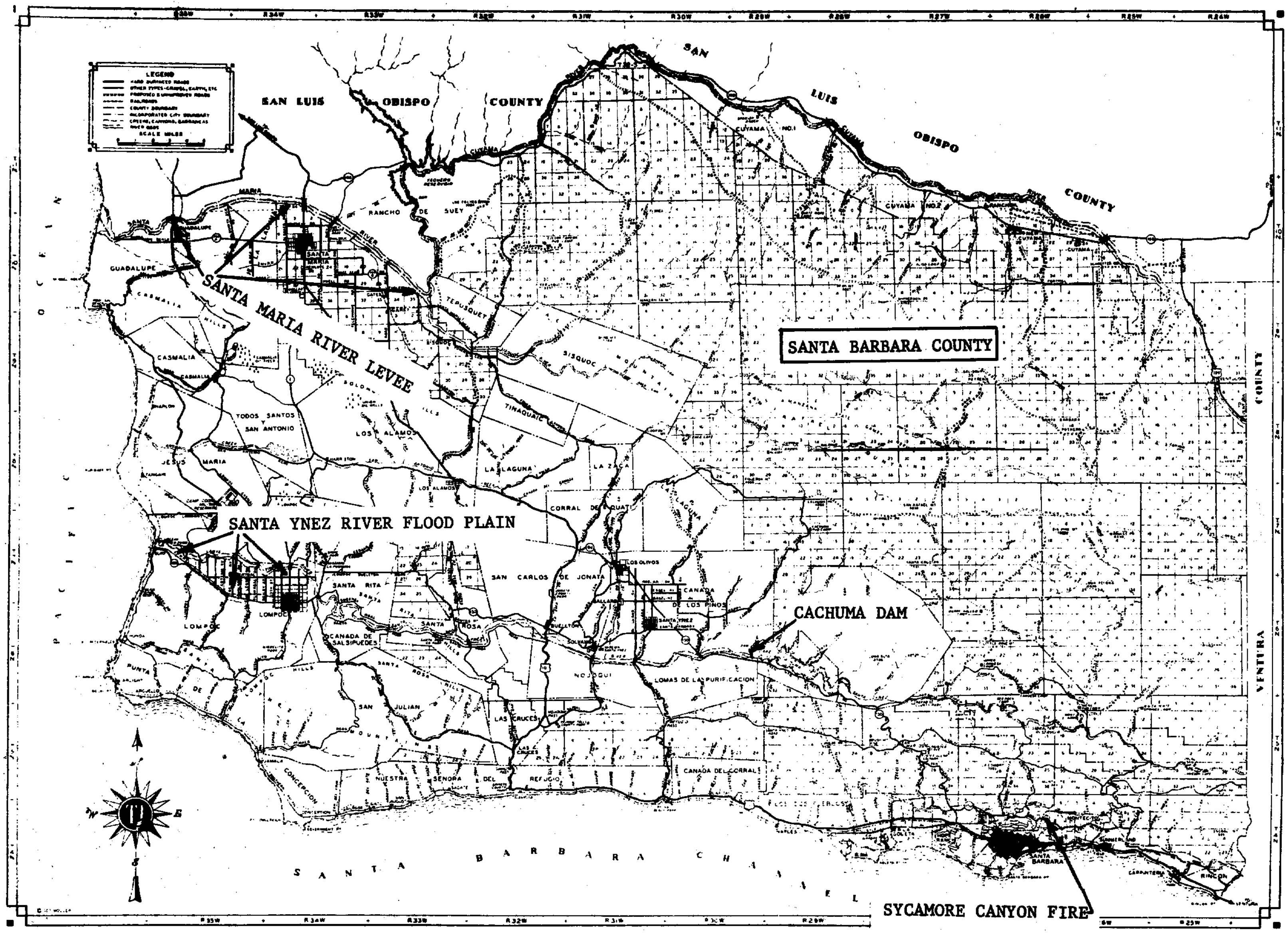


FIGURE 1 Map of Santa Barbara County.

River levees to protect the Santa Maria Valley. The levees are located on a deep sandy alluvium and consist of an earth embankment protected from erosion by a rock facing 18 in. thick on the river side. This rock facing extends to a depth of 15 to 16 ft below the river bed (Figure 2).

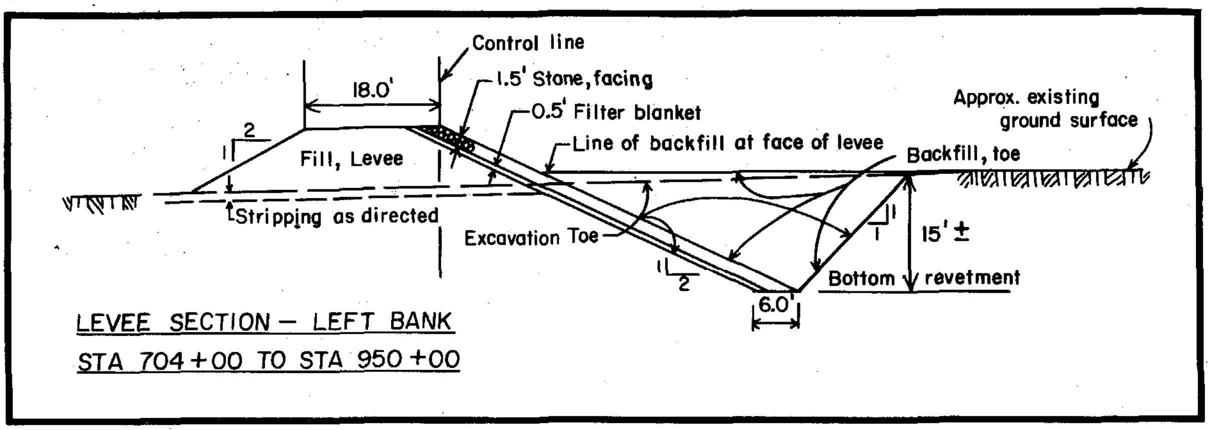


FIGURE 2 Santa Maria River levees.

In order to construct the rock facing below ground, it was necessary to excavate and backfill a trapezoidal prism on the river side. In many reaches the native soil had some degree of resistance to erosion and sediment transport. The backfill has virtually no cohesion and is easily eroded.

The levees are designed for a standard project floodflow of 150,000 to 160,000 cu ft/s. The maximum flows experienced since their completion have been 30,000 cu ft/s in 1966, 25,000 cu ft/s in 1969, 22,000 cu ft/s in 1978, and 8,000 cu ft/s in 1980. The levees have performed as planned as long as the main currents of flow in the river were parallel with the levees. There have been problems of severe undercutting and levee erosion, however, where the main currents have impinged on the levee at an appreciable angle. This was first experienced in 1966 near the downstream end of the south bank levee, where the main current crossed from north to south and impinged on the levee at 80 degrees. The resulting extreme curvature of streamlines caused superelevation of the water surface and spiral flow, which undercut the rock facing and caused two crescent-shaped slumps of the riprap (Figure 3).

Similar, but more severe, problems were experienced at several locations on the levees in 1969. The worst problem was just downstream of the Bradley Canyon confluence east of Santa Maria, where the rock facing was lost for several hundred feet and a breach was prevented only by prompt flood fighting efforts. The dozing of earth against the back side of the levee and the dumping of large rock (Figure 4) into the eroded area were effective in maintaining the integrity of the levee. A breach at this location would have resulted in flooding of the city.

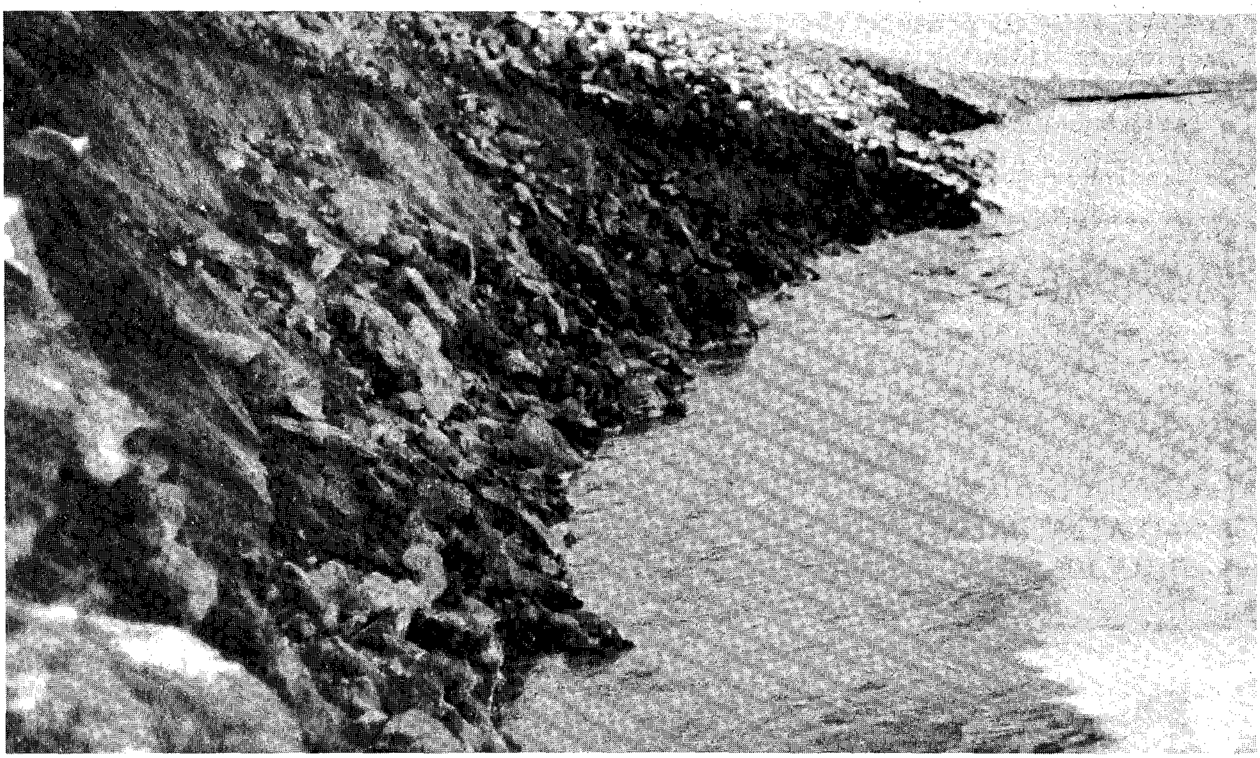


FIGURE 3 Slump in south levee of the Santa Maria River resulting from flow impinging bank at approximately 80 degrees during 1966 flood.

The problems with the levee demonstrated the need for levee patrols and preparedness for emergencies. A Telemark telemetry unit was added to an existing stream gage just upstream of the levees so that gage heights could be determined by telephone. Levee patrol teams, including personnel from several county departments, were organized and trained. Rock was stockpiled so that it would be available when needed. Arrangements were made with the City of Santa Maria and private contractors for quick response to calls for men and equipment. The city organized civil defense planning based on a levee failure.

A major item in the training of levee patrollers was the recognition of current impingement. Main currents in the river are visible by the light-colored foam that invariably exists. When current impingement is observed, one patroller goes down the face of the levee with a safety rope to check for missing rock at the water surface. It is almost impossible to observe this from a patrol vehicle, because the view of the patroller is tangent with the rock facing (Figure 5). Failures begin at the toe of the rock, some 15 ft below the surface, and progress upward like a rising moon. By the time a failure is visible, it is already a serious problem.

In 1980 a patrol did discover a failure at the Bradley Canyon confluence at 1 a.m. Emergency plans were put into action and emergency repair work was completed by 7 a.m.

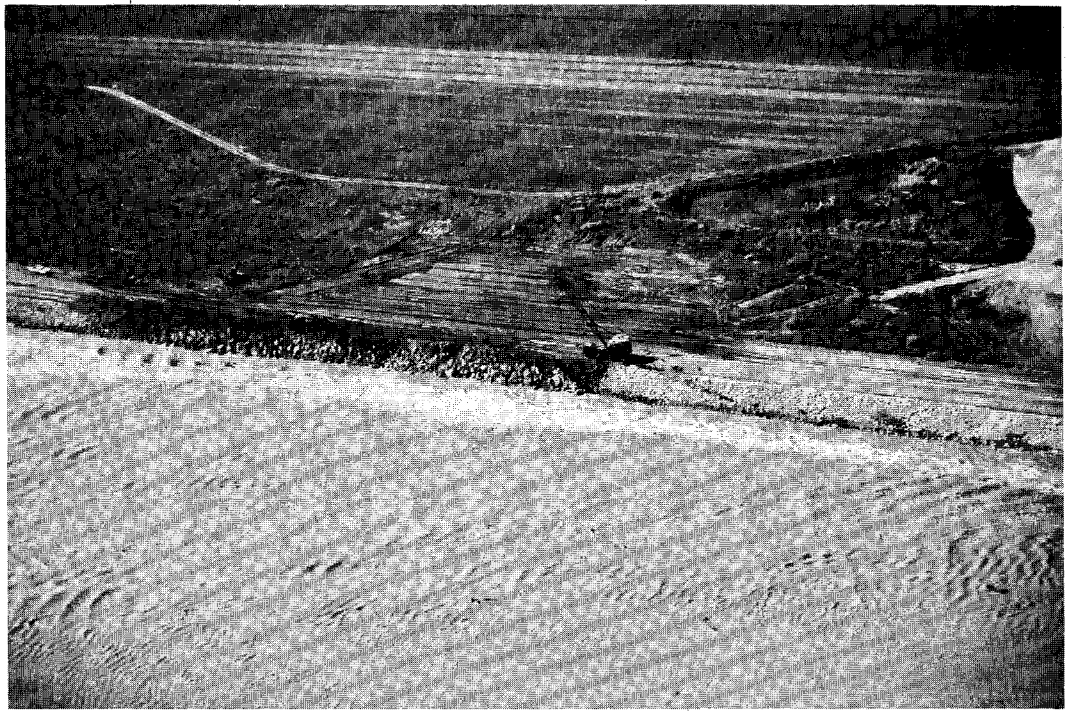


FIGURE 4 Rock being dumped on face of levee just downstream of Bradley Canyon to prevent breakout and flooding of Santa Maria during 1969 flood.

FIGURE 5 Impingement of flow on dike.

In summary, it is believed that the most important factors in dealing with the Santa Maria River levee problems are:

1. Awareness of the problem
2. Twenty-four-hour levee patrols when flows exceed 5,000 cu ft/s
3. Having rock stockpiled for emergencies
4. Being able to doze earth from the area behind the levee

The permanent solution to the levee problems is the rehabilitation project that the Corps of Engineers will construct. It consists mainly of building rock groins perpendicular to the levee in areas of current impingement. These groins will prevent high-velocity flow adjacent to the levee. Construction is expected to begin in early 1981.

SANTA YNEZ RIVER FLOOD WARNING SYSTEM

The Santa Ynez River flows from the Ventura County line on the east to the Pacific Ocean at Vandenberg Air Force Base on the west. The upper watershed receives the highest rainfall amounts in Santa Barbara County. There are three dams in the watershed: Juncal and Gibraltar are small (9,000 acre-ft) and Cachuma is large (205,000 acre-ft). None has dedicated flood control storage. The river is fairly well confined upstream of the Lompoc Valley. There is a major floodplain west of the City of Lompoc, which includes some facilities of South Vandenberg Air Force Base.

In 1969 there were two major floods that inundated the Lompoc Valley (Figure 6), causing damage in the millions of dollars. People were trapped by the rising waters, and there were many rescues (Figure 7). It was apparent that people and livestock could have been evacuated in advance of the flooding and that property damage could have been partially mitigated if there had been timely and believable early warnings of impending flooding. Therefore the Santa Barbara Flood Control and Water Conservation District established the Santa Ynez River Flood Warning System.

A flood warning system consists of three main elements.

1. Warning origination
2. Warning dissemination
3. Response and action by receivers of warnings

The Flood Control District installed a telemetry system to be used for early warnings and hydrologic data collection. There are rain gages at several locations, water level sensors on the three reservoirs and the river, and spillway gate position indicators on Gibraltar and Cachuma. These sensors are connected to 16-channel digital encoders and sending units that were custom designed and built to district specifications. The data are sent at very high rates over phone lines or microwaves to the district office, where the data are decoded and either printed on a teletype or fed into a minicomputer for processing. Inflow and outflow at the three reservoirs are calculated directly from the data. These flow data alone allow a six-hour flood warning for the Lompoc Valley.

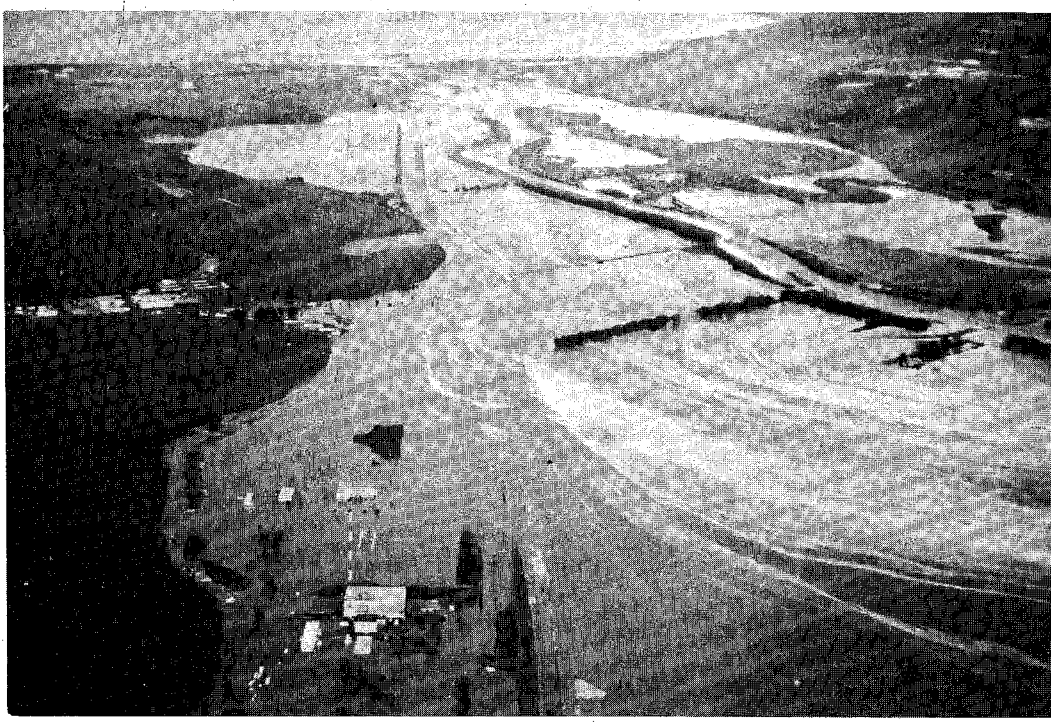


FIGURE 6 Inundated areas in Lompoc Valley by flooding from Santa Ynez River in 1969.

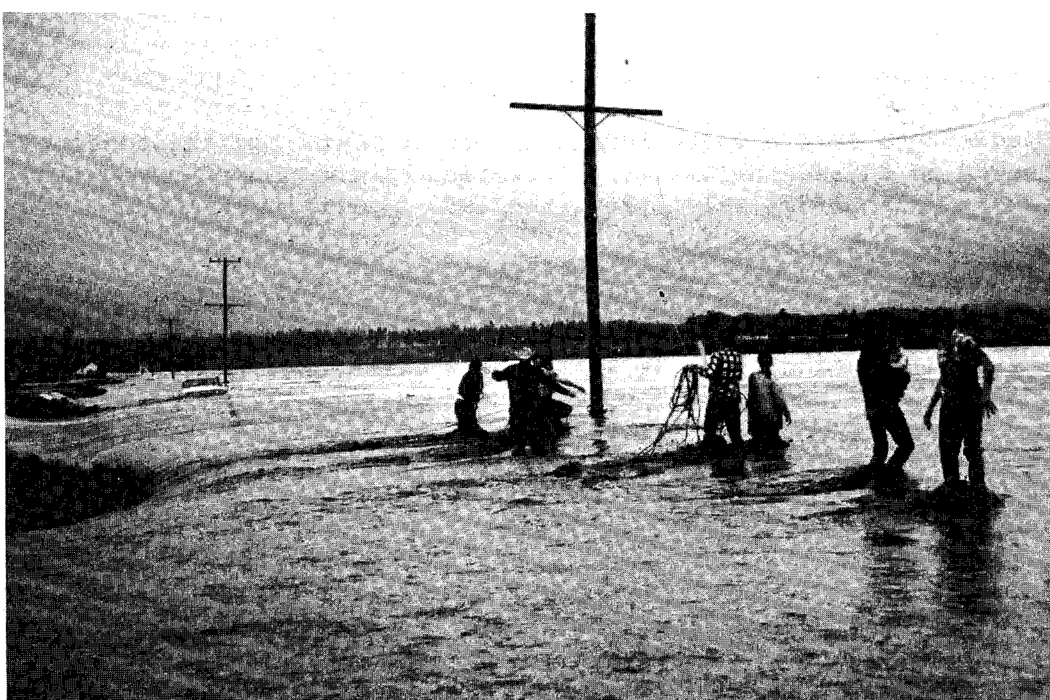


FIGURE 7 Rescue operations in Lompoc Valley during 1969 floods. There are still children in the car. One has already been rescued.

A computer model of the Santa Ynez watershed was developed to predict runoff from telemetered rainfall data (Figure 8). The runoff volume is calculated using an antecedent index method developed by the National Weather Service River Forecast Center in Sacramento. The runoff is distributed by the unitgraph method, rigorous reservoir routing, and channel routing. The model allows an additional 6-hour advance warning time, for a total of 12 hours. If used with predicted rainfall, a 24-hour warning can be given. However, experience has shown that quantitative precipitation forecasts are not sufficiently accurate for meaningful warnings.

The dissemination phase of the system includes a procedures manual, standardized warning messages, a map showing areas inundated at various flow rates, and a communication plan with backup systems. The emergency planning and implementation is the responsibility of the agencies that receive the flood warnings.

The system received its first real test in the 1978 storms. Because of the drought in the preceding years, a program of cloud seeding to increase rainfall was initiated in December 1977. Cachuma Reservoir was 75,000 acre-ft below full when the February 18 storm occurred. After 9 in. of rain was measured by the telemetry, computer forecasts indicated that the reservoir would fill and seeding was terminated. Cachuma did indeed just fill, but the spill was very minor and no flooding occurred. In the early evening of March 3 a forecast of 5 in. was received from the district's private weather consultant. A computer forecast with this rain indicated a major spill from the already full Cachuma, and an advisory message was issued. Overnight, 7 to 8 in. of rain actually fell, and flood warnings were issued at 8 a.m., March 4, for a flood of 50,000 cu ft/s that evening. The warnings were quite accurate, and effective advance measures were taken. Figure 9 shows the observed and forecast inflow into Lake Cachuma.

Subsequent to 1978 a faster computer was acquired and the forecast model was extended to include all major tributaries of the Santa Ynez River. The system functioned effectively during the 1980 storms.

SYCAMORE FIRE EMERGENCY FLOOD PROTECTION WORK

In July 1977 an intense wildfire burned some 320 homes and the Sycamore Canyon watershed in and adjacent to the City of Santa Barbara. Experience following previous fires indicated that serious flooding and debris flows usually result from rains on newly burned watershed (Figure 10). Therefore. the district immediately began planning for emergency flood protection measures in and downstream of the burned area.

The first step was to determine and map problem areas, including areas subject to flooding, mudslides, debris movement, and so forth. Roads that would probably be closed during storms were identified so that alternative routes could be planned. Inadequate bridges and culverts were identified. Possible debris dam and check dam sites were located. The maps were used by public safety agencies to plan evacuation and rescue activities, by the district to plan flood control measures, and for public information at meetings with homeowner groups.

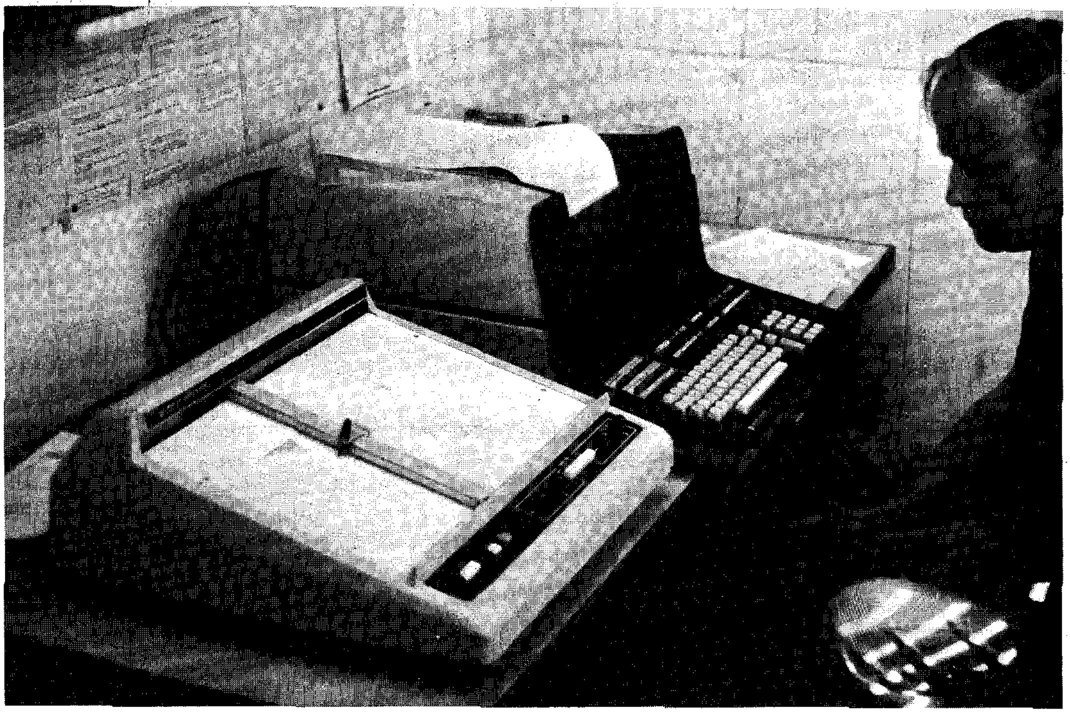


FIGURE 8 Desktop computer used in forecasting floodflows in Santa Ynez River.

Sprinkler tests on burned slopes were conducted to determine runoff potential. The results were compared with the effects of previous fires.

A booklet telling homeowners how to protect themselves was distributed. The district supplied sandbags and sand at various locations in the burned area and gave technical advice on individual protection (Figure 11). Grass seed and spreaders were furnished to homeowners willing to seed and irrigate burned slopes in advance of the rains. The availability of flood insurance was publicized.

A local analog rainfall and stream level telemetry system was installed, and a flood forecasting program was devised. Flood hazard conditions were categorized as green, yellow, or red and were given to residents by means of radio broadcasts and a flag flown at the entrance to the canyon. Forecasts were also supplied to city and county emergency operation centers.

Revegetation of burned areas was an important element of the plans. Certain critical slopes were hydromulched. This process, which consists of spraying a waterborne mixture of seeds, fertilizer, and wood fibers (mulch), is expensive but effective. The balance of the burn was seeded and fertilized by helicopter (Figure 12). The aerial seeding was delayed until just before the first predicted rainfall. It was important to apply the seed before a crust formed on the ash but as late as possible to avoid loss of the seed to the wind and birds.

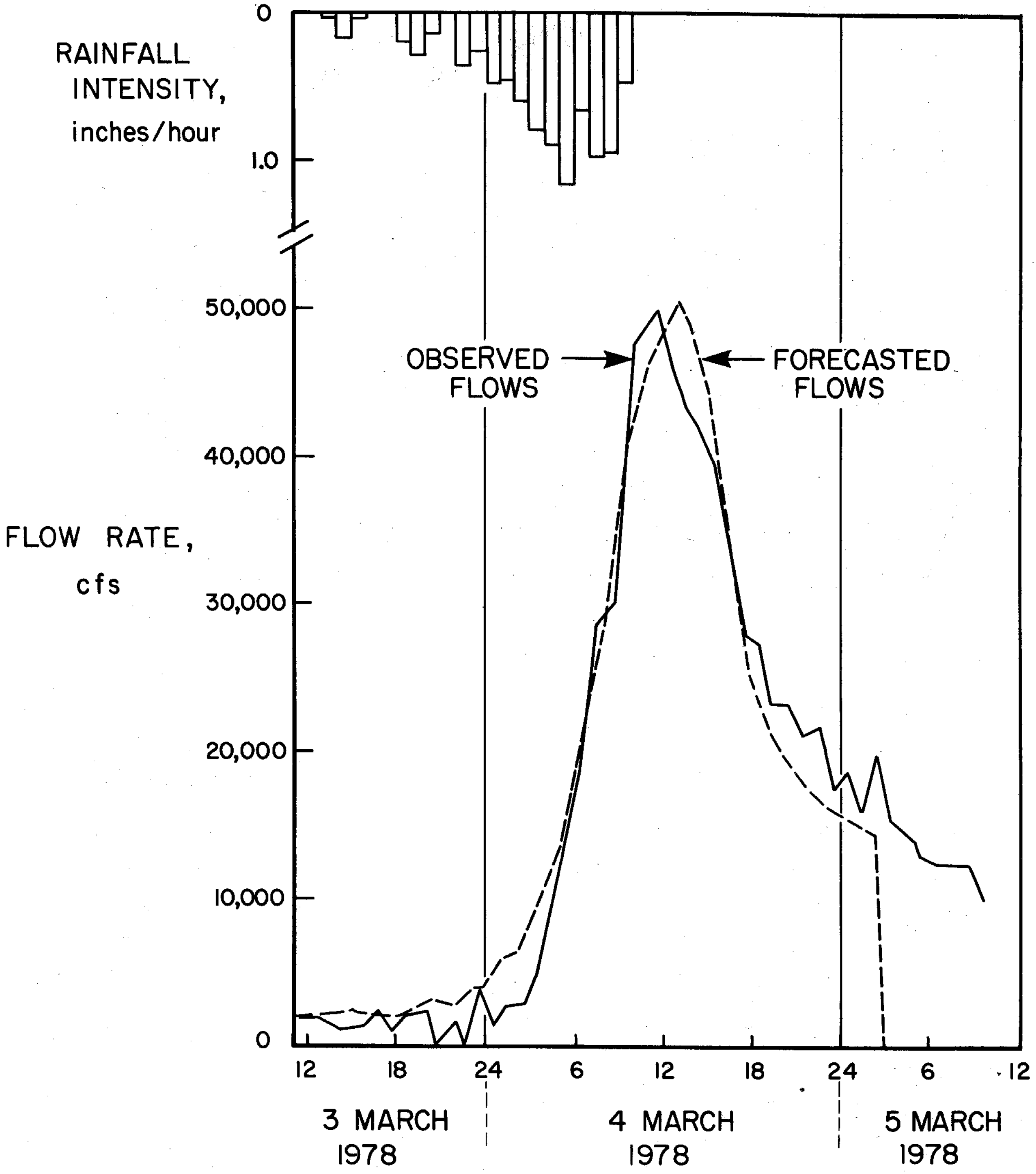


FIGURE 9 Observed flow rate into Lake Cachuma during flood of March 1978 compared with flow forecasts based on rainfall data.

FIGURE 10 Damage from flood resulting from rain on a recently burned watershed.

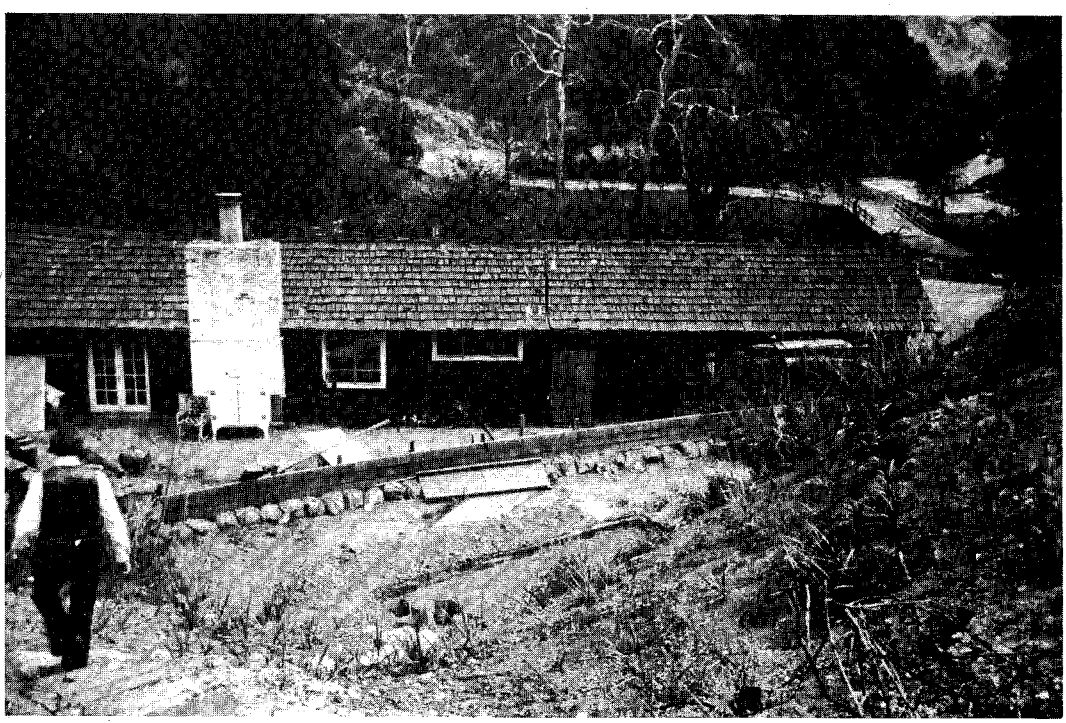


FIGURE 11 Wood wall and sandbags installed with advice of the Flood Control District in anticipation of flooding following a fire.

FIGURE 12 Seeding and fertilizing burned watershed by helicopter.

Structural measures included culvert replacement by road agencies, removal of bridges with inadequate waterways, clearing and enlarging Sycamore Creek, protecting raw banks with pipe and wire revetment (Figure 13), and constructing grouted rock check dams and debris barriers (Figures 14 and 15).

The emergency measures were accomplished in about eight weeks with concurrent design, right-of-way, and construction activities. The construction was done through a combination of force account, equipment rental, and contract. The total cost was $225,000, which was shared equally by the district and the state under Section 128 of the Water Code. (Federal funds were theoretically available, but paperwork requirements would have delayed implementation into the rainy season.)

The 1977-78 rainy season was very wet, with several intense storms. There was heavy runoff from the Sycamore burn but no major flooding. The emergency protection measures seemed to work very well. The district cannot claim total responsibility for the low damage figures, but we like to believe that our expenditures had a very high ratio of benefits to costs.

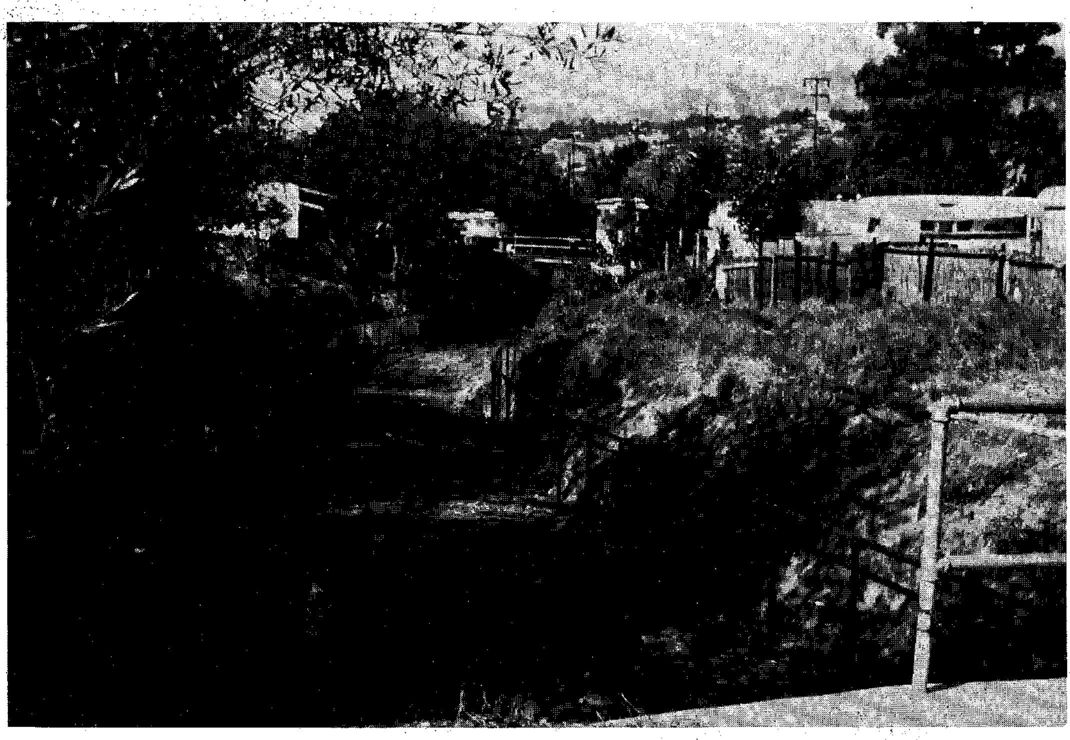


FIGURE 13 Pipe and wire revetment installed in Sycamore Canyon as protection against floods anticipated after fire of July 1977.

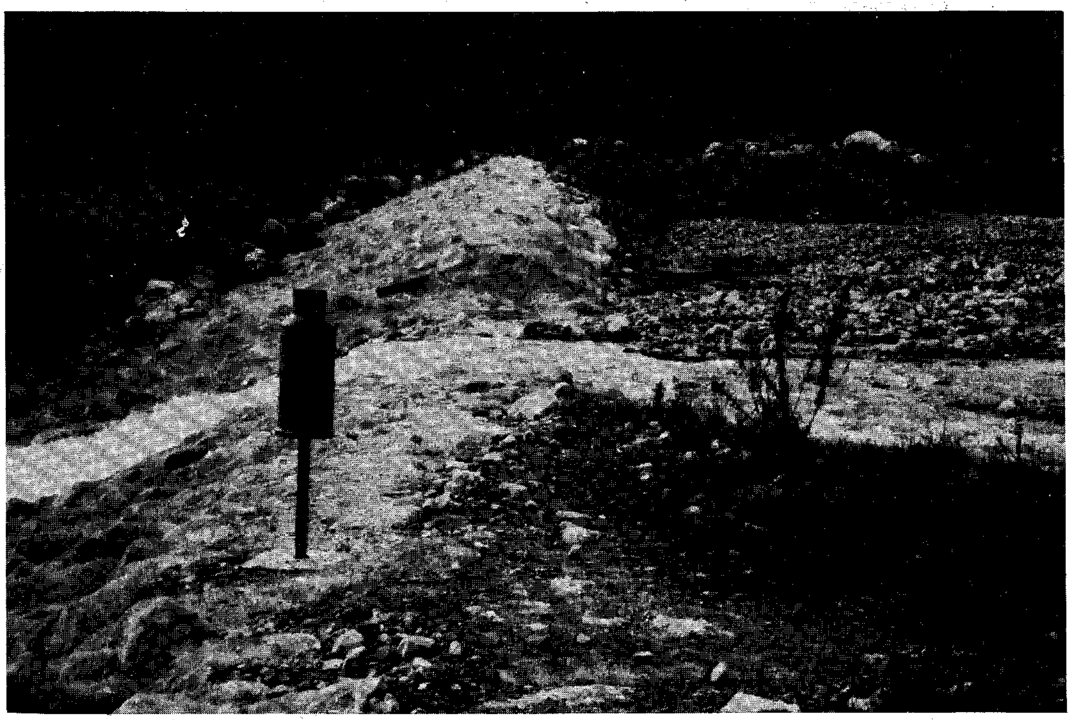


FIGURE 14 Grouted rock check dam installed in Sycamore Canyon after fire of July 1977.

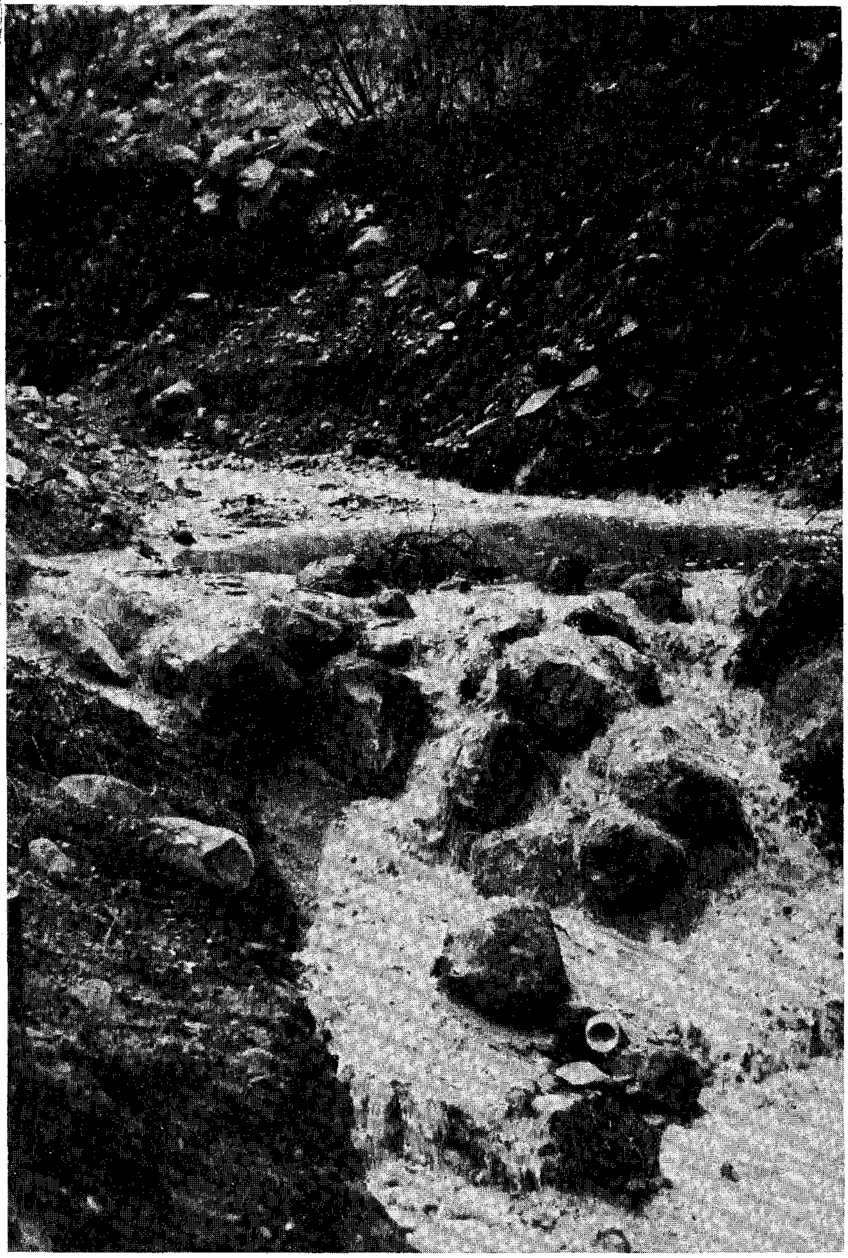


FIGURE 15 Grouted rock check dam installed in Sycamore Canyon after fire of July 1977.

EROSION AND DEPOSITION AT A SAND AND GRAVEL MINING OPERATION IN SAN JUAN CREEK, ORANGE COUNTY, CALIFORNIA

by Vito A. Vanoni, Robert H. Born, and Hasan M. Nouri

Sand and gravel have been mined from a 2-mile reach of San Juan Creek, about 6 miles upstream of the City of San Juan Capistrano, for over a decade. The reach in which mining occurred is immediately downstream of the Caspers Regional Park of Orange County. The mining proceeded upstream and the rate of gravel extraction increased with time. In 1965 the mining pit was over 1 mile from the park boundary and 15 ft deep. In 1976 the upstream edge of the pit was approximately 120 ft from the park and 60 ft deep.

The only appreciable flows in San Juan Creek from 1965 to the fall of 1976 occurred in the winters of 1965 and 1969, which produced peak flows of 4,000 cu ft/s and 22,000 cu ft/s, respectively, at a gaging station approximately 5 miles downstream of the park. The 1969 flood is reported to have completely filled the preflood pit in the creek bed with sediment carried by the flood. This pit was approximately 3,500 ft downstream of the park, and no erosion in San Juan Creek occurred in the park area.

Unusually high flows occurred in the three water years 1977-78 through 1979-80. These flows deepened and widened the creek channel upstream of the gravel pit and raised the bottom of the pit by depositing most of the sediment carried into it.

Data are given on rainfall, runoff, and erosion upstream of the gravel

---

Vito Vanoni is Professor of Hydraulics, Emeritus, at the California Institute of Technology in Pasadena, California. Robert Born is President of Born, Barrett & Associates in Newport Beach, California. Hasan Nouri is Senior Engineer with Camp Dresser & McKee, Inc., Newport Beach, California.

Note: Special appreciation is extended to Herbert Harger of the Conrock Company and to Jim Williams and Joe Natsuhara of the Orange County Environmental Management Agency for their helpful assistance and encouragement to undertake this paper. Appreciation is also expressed to M. Gamal Mostafa, Head of the Department of Civil Engineering at California State University, Long Beach, and to Camp Dresser & McKee, Inc., for use of the results of their previous investigations concluded in 1978 (Camp Dresser & McKee, Inc., 1978a,b; Mostafa, 1978).

pit and deposition in the pit starting from the fall of 1977. The progress of erosion is given by profiles and cross sections, and total quantities of erosion and deposition are given. The mechanics of the process is discussed.

INTRODUCTION

This paper presents detailed information on the impacts of the January-March 1978 floods on a sand and gravel mining operation along San Juan Creek in Orange County. Headward erosion in the upstream tributary channels and deposition in the mining excavation are described and evaluated as a result of the fortuitous availability of topographic surveys accomplished between the storms.

The paper is organized in three sections. The first section briefly describes the history of the mining operation and the tributary watershed. A second section presents information on the local hydrology of the 1978 storms. A concluding section presents an analysis of the events with particular attention to the evaluation of channel slopes and the quantities of erosion and deposition. The objective of the paper is to present a case study of channel erosion and sediment deposition rates and their relation to runoff. It is hoped that the presentation of such quantitative data will be helpful in other similar cases.

HISTORY OF MINING

Sand and gravel have been mined from a 2-mile reach of San Juan Creek, about 6 miles upstream from the City of San Juan Capistrano, for over a decade. Most of the early activity was concentrated at the lower site (see Figure 1). By the fall of 1977 the excavation had been extended to the upper site, shown in Figures 1 and 2.

Caspers Regional Park, an Orange County-owned facility, is located immediately above and adjacent to the lease site. By 1965 the lower pit was about 1 mile from the park and about 15 ft deep at its deepest point. Material was processed at a plant located adjacent to the site. By the end of 1977 the upper pit had been developed to a depth of 60 ft at its upper end, and to within a distance of 120 ft from the upper end of the lease. Side slopes of the excavations were limited to 1.5 to 1 horizontal to vertical.

Description of Tributary Area

Figure 1 also shows the extent of the three principal watersheds tributary to the upper mining site, which will be the principal focus of this paper. The three watersheds comprise a total area of nearly 77 square miles, the majority of which falls within Cleveland National Forest. San Juan Creek, the largest, covers 55.6 square miles. Bell Canyon covers 17.2 square miles and Verdugo Canyon, the smallest, covers only 4.0 square miles. Elevations range from about 360 ft above sea level in the streambed at the park boundary to approximately 4,500 ft in the upper reaches of the watershed, with most of the upper watershed lying above 2,500 ft.

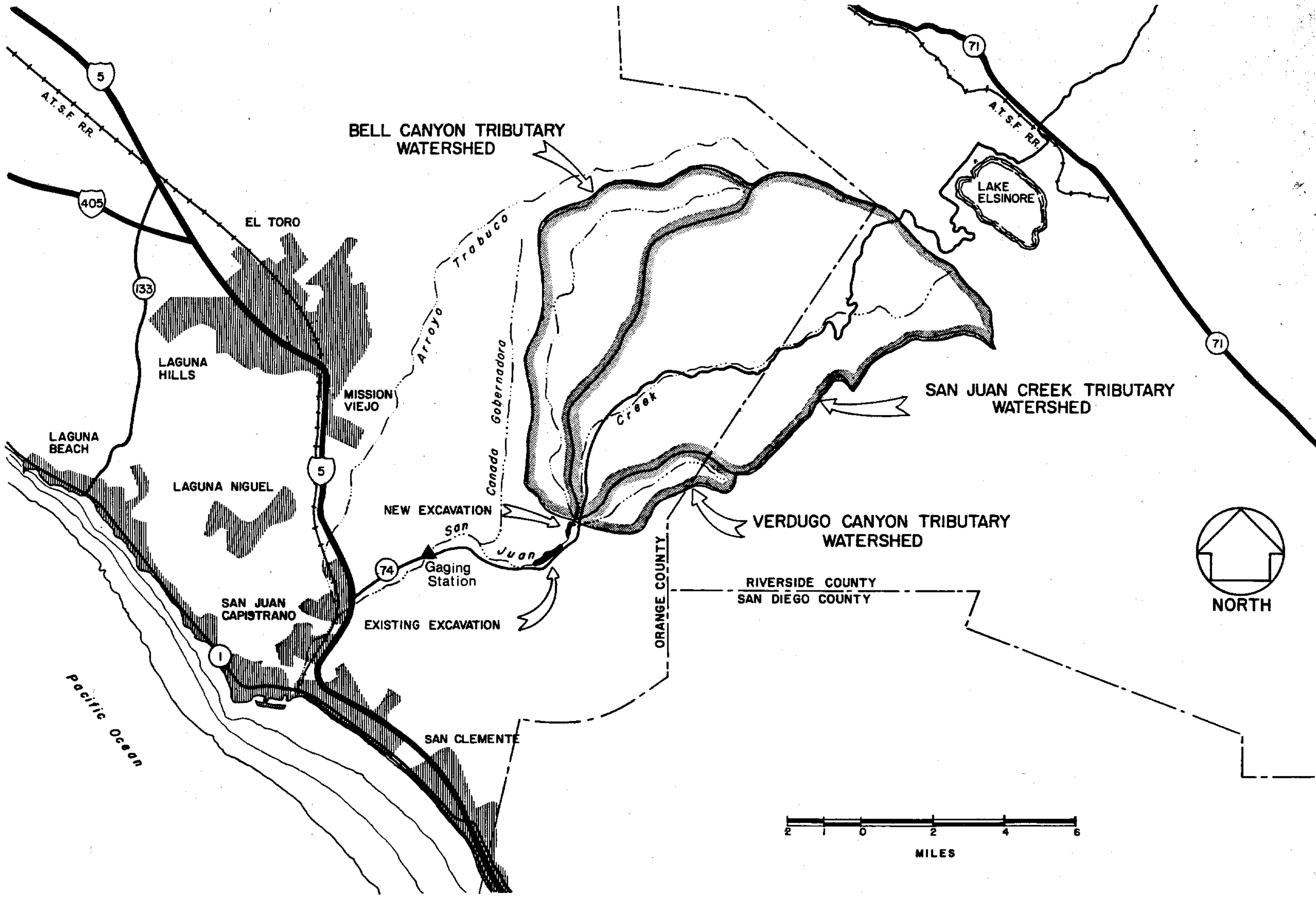


FIGURE 1 Location map of mining operation.

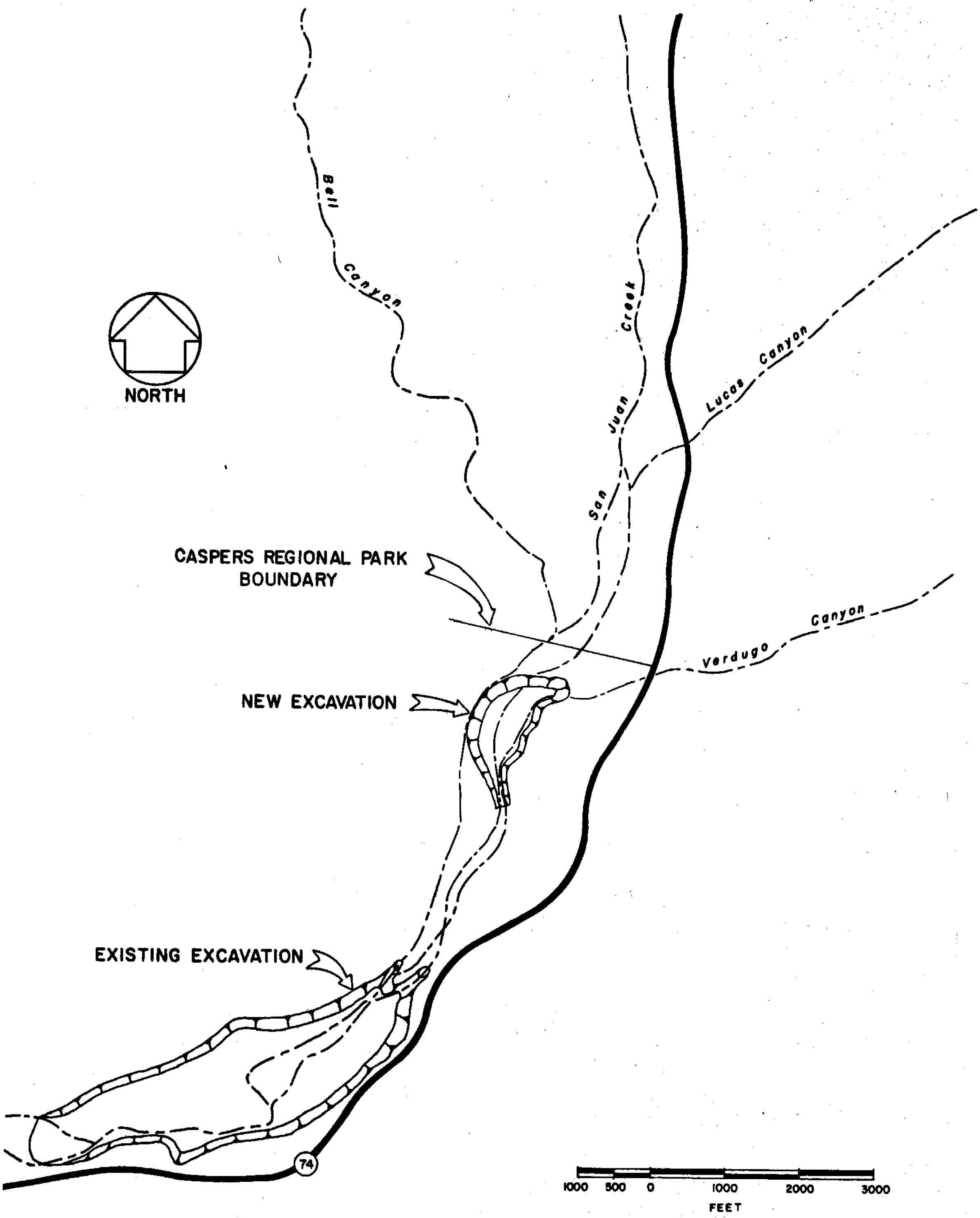


FIGURE 2 Vicinity map of mining operation, showing upper and lower excavation sites.

The tributary watersheds are underlain by a complex of igneous, metamorphic, and sedimentary rocks and have a history of frequent fires. Numerous landslides are found throughout the area, with residual bedrock landslide debris covering more than 3.7 square miles within the San Juan Creek watershed alone. It has been estimated that more than a billion tons of landslide debris are ready for transit down this one drainage area during major flood events. Bell Canyon is underlain by a wide variety of coarse clastic geologic units, including several massive landslides (Converse Ward Davis Dixon, 1978).

Large quantities of loose surface materials are available in rockslides and mudslides or otherwise generated through natural erosion processes. The upper San Juan drainage produces a high percentage of coarse sand and gravel, principally because of the hard, coarse source materials, and Bell Canyon produces large quantities of even coarser material.

Portions of the upper watersheds and deeper canyons are heavily forested, and much of the entire watershed contains a dense cover of chaparral. Scattered oak trees are found throughout the lower and medium elevations of the tributary area.

RAINFALL AND RUNOFF

At the San Juan Creek guard station, a short distance above Caspers Park, rainfall intensities of 0.4 to 0.8 in./hr are common. Normal annual rainfall within the watershed ranges from approximately 14 in. in the mining area to approximately 25 in. in the upper reaches of the tributary drainage area. On March 4 a maximum hourly rainfall of 1.9 in. was recorded, suggesting that this storm may have had a return period in excess of 100 years for one-hour, two-hour, and three-hour durations.

Our focus of attention will be on the three storms of January 15-20, February 9-13, and March 4, 1978. While records are only partially available due to operating problems at key recording gages, some indication of the storms' magnitudes can be obtained by examining the rainfall record at nearby Santiago Peak.

During the February storm a total of 5.7 in. fell during an 18-hour period at Santiago Peak, with five consecutive hours of 0.4 in./hr or more. The February storm produced 9.9 in. of rain, with 3.4 in. falling within a five-hour period. Hourly records at this station are not readily available for the March storm, but daily totals show that a total of 2.21 in. fell on March 4 and 0.58 on March 5, for a two-day storm total of 2.79 in. A summary of rainfall occurring at Santiago Peak during the three storms is presented in Table 1.

Hydrographs during the three major 1978 storms are presented in Figure 3. As shown, the peak flows in the major storms of 1978 became progressively larger, with the largest peak discharge, of 11,600 cu ft/s, occurring on March 4. These hydrographs are based on measurements by the U.S. Geological Survey at a gaging station on San Juan Creek at San Juan Capistrano about 0.2 mile

TABLE 1 Recorded Rainfall, 1978 (in.)

	Santiago Peak							San Juan Guard Station[a]
Hour Ending	Jan. 14	Jan. 15	Jan. 16	Feb. 8	Feb. 9	Feb. 10	Mar. 4	Mar. 4
1 a.m.		0.4			0.3	0.2		
2		0.5			0.4	0.1		
3		0.1			0.1	0.3		
4					0.9	0.1		
5					0.8			
6		0.1			0.5			
7					0.8	0.1		0.1
8					0.4	0.1		0.1
9						0.2		0.2
10	0.1				0.1			
11	0.1				0.1	0.1		0.1
12 noon	0.1	0.1				0.3		0.1
1 p.m.	0.1		0.2		0.1			0.1
2	0.1	0.1	0.1		0.1			0.5
3	0.3		0.1		0.1		No hourly record	1.9
4	0.2		0.4		0.4			0.1
5	0.1		0.2		0.3			
6	0.1		0.5	0.1	0.3			
7	0.1		0.5	0.1	0.1			
8	0.2			0.2	0.4			0.1
9	0.3			0.1	0.2	0.1		
10	0.4		0.1	0.3	0.2			
11	0.5			0.2	0.2			
12 p.m.	0.4			0.2	0.3			
Totals	3.1	1.3	2.1[b]	1.2	7.1	1.6	--	3.3

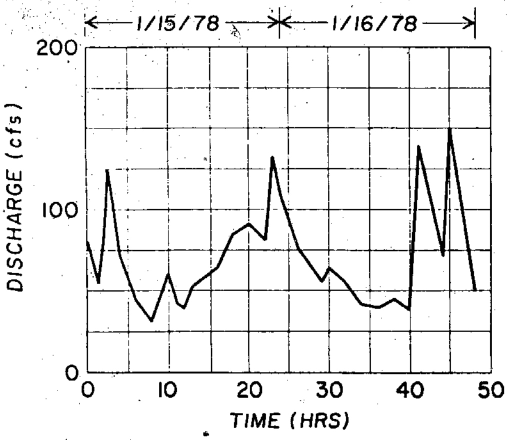
[a]Record began in March 1978.

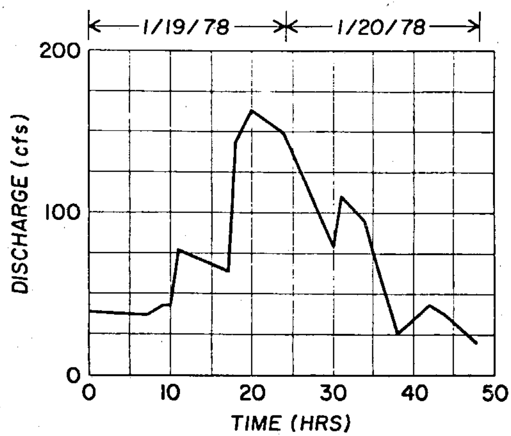
[b]Rainfall record at Santiago Peak; 0.9 in. on January 19, none on January 20.

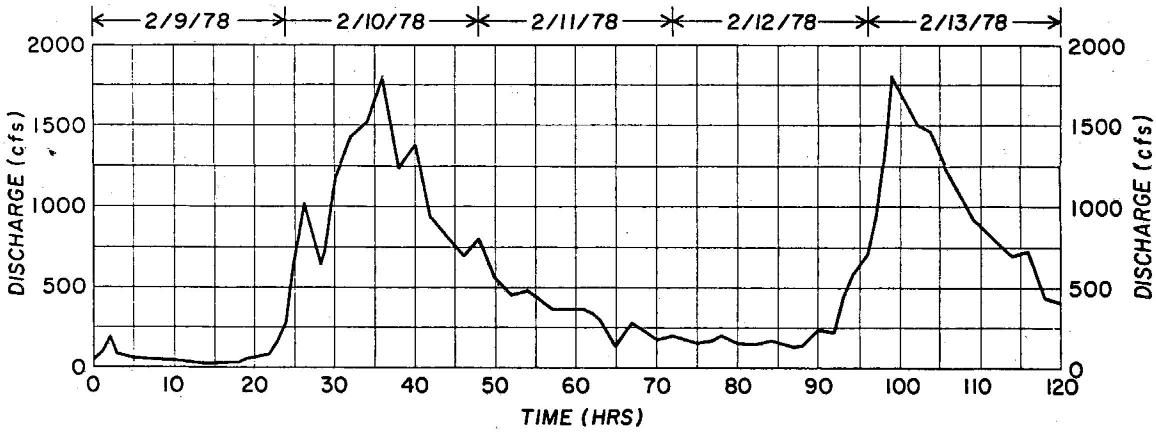
upstream of its confluence with Arroyo Trabuco. The drainage area of San Juan Creek at the gaging station is 117 square miles, compared with 77 square miles immediately downstream of the confluence with Verdugo Canyon (at the new mining site). Hydrographs obtained from the gaging station were transposed to the study area by means of methodology suggested by Creager et al. (1947).

EROSION AND DEPOSITION

As a result of the three major storms occurring in 1978, the channels of San Juan Creek, Bell Canyon, and Verdugo Canyon upstream of the sand and gravel excavation were subjected to severe headward erosion, with the concurrent formation of a delta of eroded material at the upper end of the







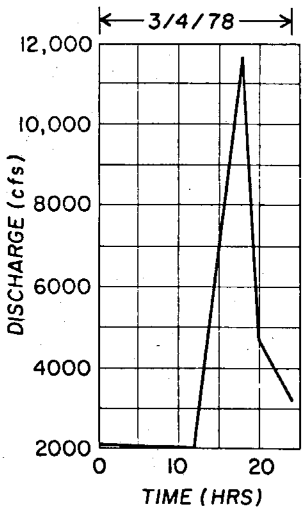


FIGURE 3 Major 1978 storms in San Juan Creek downstream of its confluence with Verdugo Canyon.

excavation. During each flood event headward erosion proceeded up each of the three channels while the delta in the excavation became progressively larger, eventually forming a control against further vertical scour immediately upstream of the former pit boundary. Thereafter, as the channel bottoms were elevated to conform to the configuration of the delta, the channels widened by lateral scour and bank cutting.

Figures 4-7 illustrate channel conditions following the different flood events. Figure 4 shows the eroded condition of the right bank of the San Juan Creek channel downstream of Bell Canyon on January 17, shortly after passage of a portion of the January flood. The channel cutting through the right bank was formed by a distributary of Bell Canyon. Figure 5 shows the condition of the mouth of Verdugo Canyon at the gravel pit on the same date. The delta formed by Verdugo Creek in the mining excavation has been eroded by flows from San Juan Creek, as shown by Figure 5.

Figure 6A shows the extent of exposure of a semiresistant sandstone outcropping on San Juan Creek, some 900 ft above the park boundary, on February 2. A similar extension of this outcropping was also exposed in Bell Canyon. As headward erosion proceeded, the outcropping substantially inhibited further vertical erosion at these locations. Concurrently, downstream gradients from the outcroppings flattened as the delta in the pit rose in height. Both upstream headward erosion and lateral bank cutting continued from these new controls. By April 11 the channel had eroded both vertically and laterally to expose more of the outcropping, as shown in Figure 6B. Figure 7 shows the extent of vertical and lateral erosion at the park boundary on January 17, shortly after the passage of only part of the January flood. Note the suspended fence that is located along the alignment of the park boundary, some 100 ft above the edge of the upper lip of the mining excavation.

Plans and profiles of erosion and deposition in and above the excavation following the various floods of 1978 are shown in Figure 8. These profiles and the schematic plan show the growth of the delta in the pit and the progress of degradation in the upstream channels, based on topographic surveys performed after each flood event. Line a-b, which has a gradient of 0.01, represents the natural slopes of both San Juan Creek and Bell Canyon prior to the 1978 floods.

The storms of January 1978, having peak discharges of only 160 cu ft/s or less, quickly caused major degradation, as shown by the profile g-h, which has a slope of 0.03. Storms occurring between February 9 and 13, 1978, produced further growth of the delta in the pit and a further deepening and widening of the upstream channels. The resulting slopes are shown by lines e-f and f-k, with values of 0.02 and 0.015, respectively. The March 4 flood, with a peak discharge of 11,600 cu ft/s, produced the profile represented by the line c-d-j.

The slope between points c and d flattened to 0.016, while between points d and j in the delta the slope approached the natural gradient of 0.01 that existed between points a and b prior to the occurrence of the 1978 floods.

FIGURE 4 San Juan Creek downsteam of Bell Canyon, January 17, 1978.

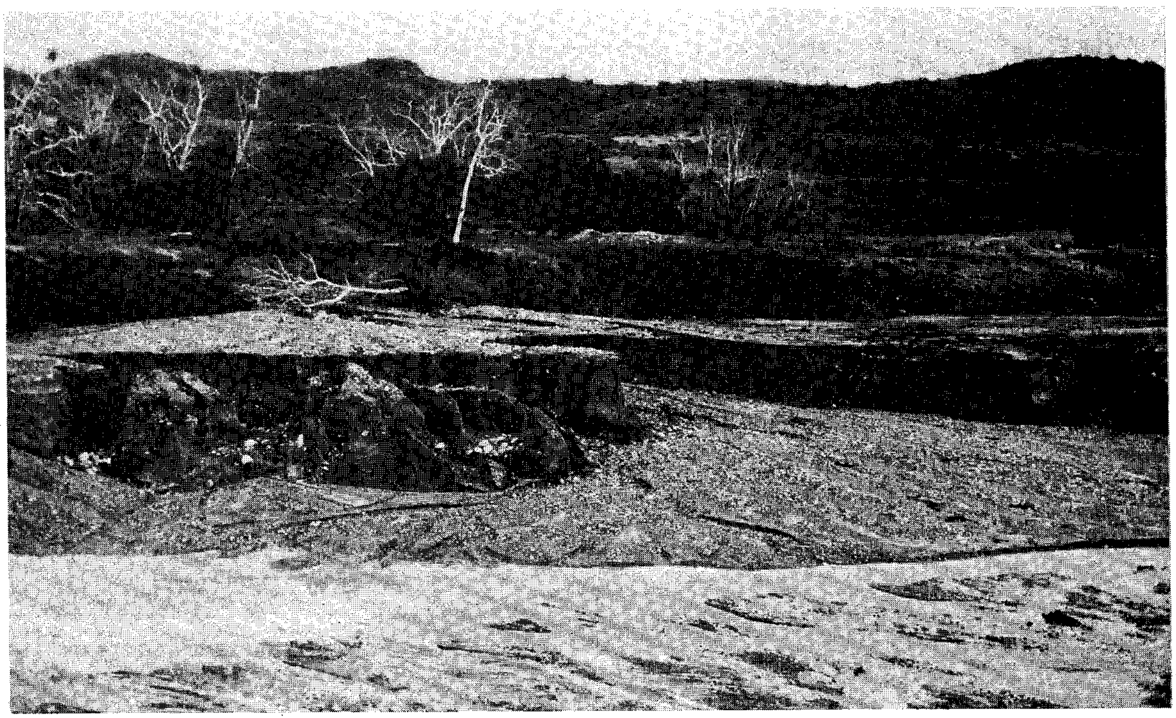


FIGURE 5 Mouth of Verdugo Canyon at mining excavation, January 17, 1978.

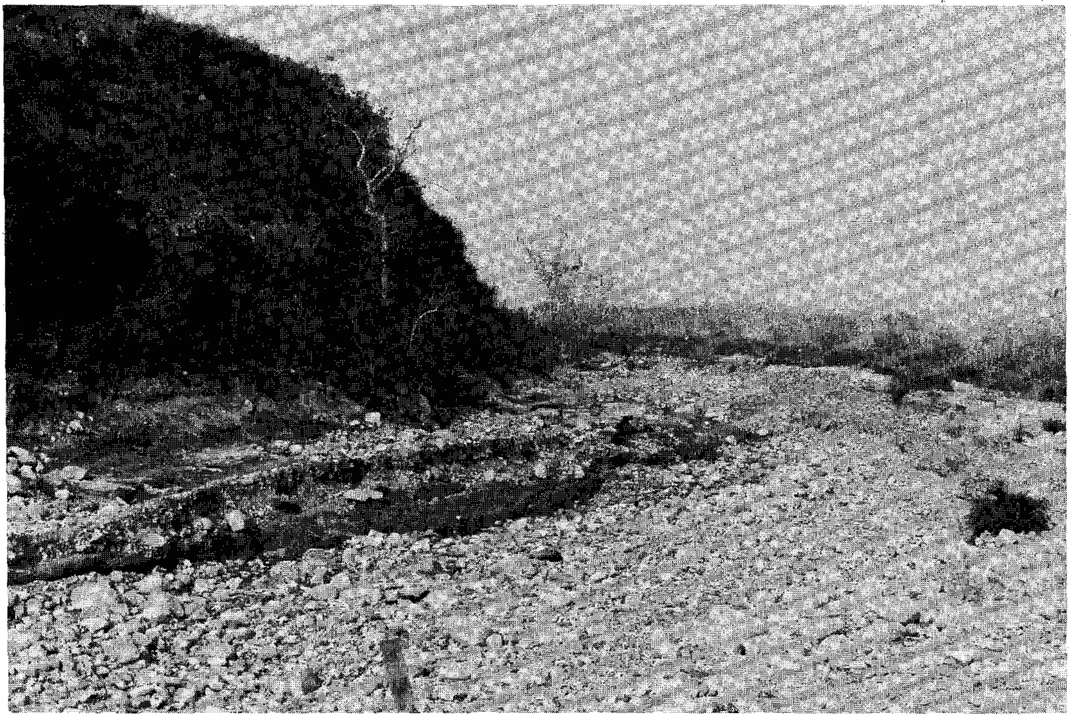


FIGURE 6A Semiresistant sandstone outcropping, San Juan Creek above Bell Canyon, February 2, 1978.

FIGURE 6B Semiresistant sandstone outcropping, San Juan Creek above Bell Canyon, April 11, 1978,

FIGURE 7 View toward right bank of San Juan Creek channel at Caspers Regional Park boundary, January 17, 1978.

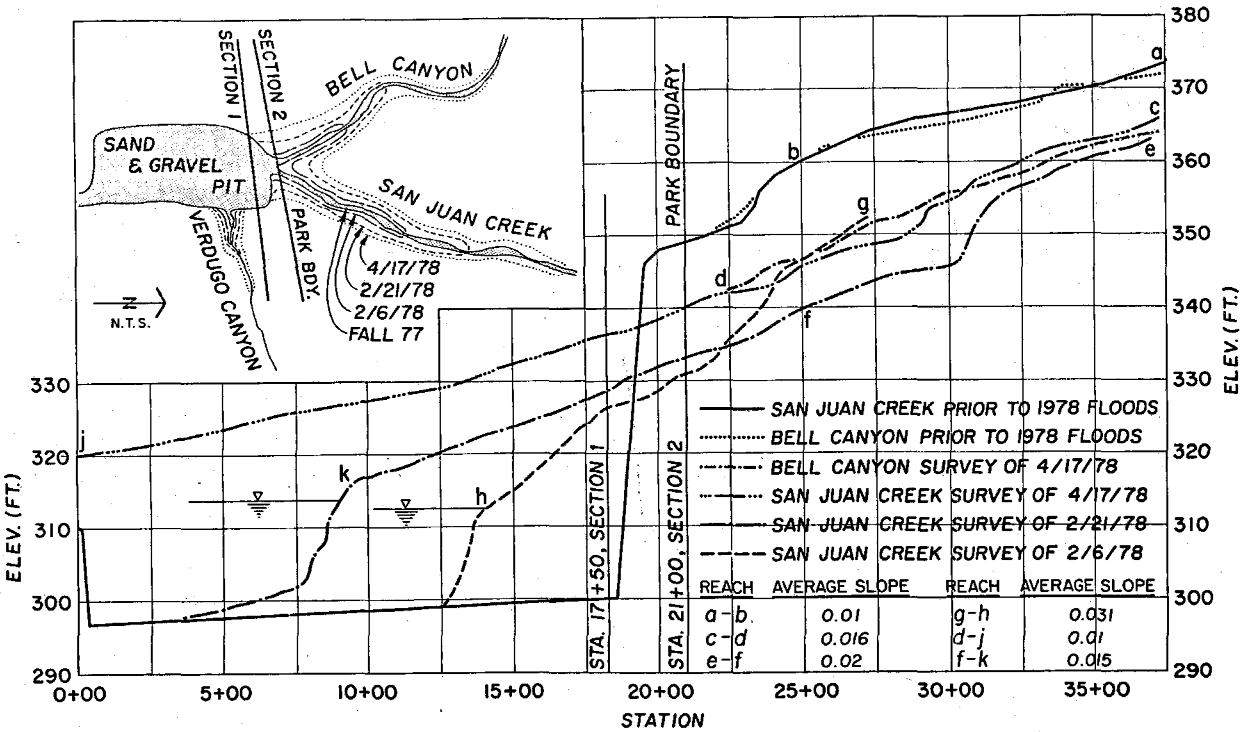


FIGURE 8 Plan and profile of San Juan Creek at upstream edge of gravel pit during floods of 1978.

Figure 8 shows the locations of cross sections 1 and 2 used in the analysis of erosion and deposition zones. Figures 9, 10, and 11 depict the progress of erosion and deposition at these sections during the three flood events.

MECHANISM OF SEDIMENT MOVEMENT

As can be noted from Figures 9-11, the channel beds remained relatively flat transversely in both the depositing and eroding sections. The deposits were built up more or less uniformly across the mine excavation or pit as the stream moved from side to side in the classical delta-building fashion. The levels in the channel upstream of the pit were controlled by the level of the deposits in the pit area when the channel was eroding as well as when it was aggrading. For this reason the channel bed remained relatively flat in section during the sequence of storms, as shown by section 2 of Figures 9-11.

The beds of the degraded creeks have coarse gravel, cobbles, and boulders on the surface, which form an armor layer and tend to limit degradation. A close view of the bed armor in San Juan Creek after the January 1978 floods is shown in Figure 12. Particles over 1 ft in maximum dimension are common at this site. Sediment of sizes comparable with those shown in Figure 12 also existed in San Juan Creek and its tributaries even before degradation occurred, indicating that armoring is common in these streams.

It is visualized that a flood degrades the streambed by removing particles that it is competent to transport, leaving the coarse particles and those shielded by the coarse ones. Presumably the maximum competency of a flood to move bed sediment coincides with the peak flow, so that as the flood and its competency wanes the bed will be armored and therefore stable. Sands and fine gravels that are transported into an armored reach of a stream will tend to be carried through the reach much as if they constituted the wash load of the flow. The bed armor is expected to persist until a flood occurs with flows exceeding the one that formed the armor. When this happens the bed will degrade, expose more of the large particles, and finally stabilize at a lower level. Hollingshead (1971) actually detected with a hydrophone the beginning of movement of bed sediment in a gravel river. He found that bed motion started at a flow of 300 cu ft/s, which on the average occurred during 30 days per year. The median size of the bed sediment was 28 mm.

The deposits in the creeks and in the pit were sampled at three different times. The difficulty of collecting representative samples can be judged from Figure 13, which is a photograph of the deposits in the pit approximately 300 ft downstream from the park after the January floods. To represent the cobbles properly one needs a very large sample. The weight of the sample shown in the bag in Figure 13, and of others, was from 10 to 15 lb. The cobbles showing on the surface of Figure 13 appear to form a small fraction of the deposit. The photograph suggests that the cobbles are part of an incipient armor resulting from erosion by the receding flood of deposits formed at higher flows.

Table 2 shows the size distribution in 10 samples of sediment deposits in the three creeks in and near the park. The subscript of d in the table gives

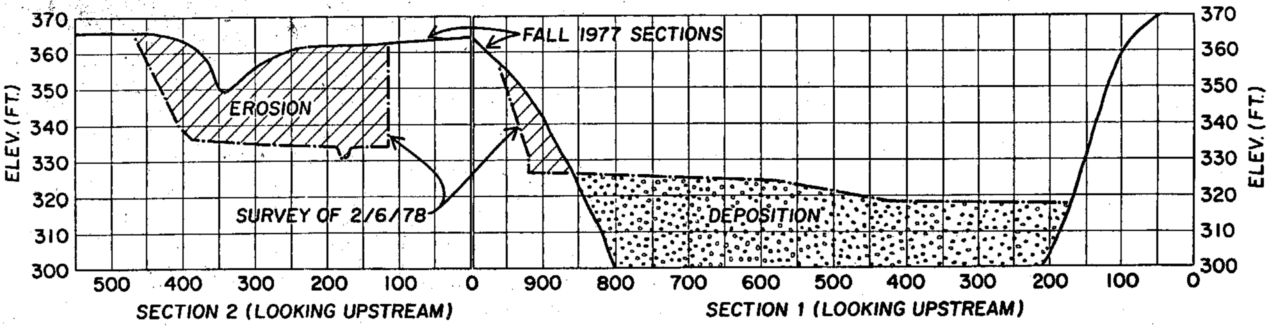


FIGURE 9 Erosion and deposition at upstream edge of gravel pit in San Juan Creek after floods of January 1978.

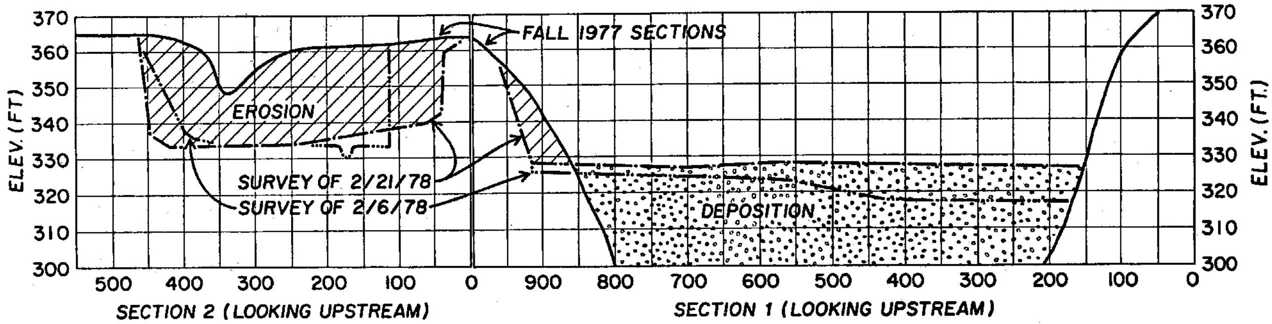


FIGURE 10 Erosion and deposition at upstream edge of gravel pit in San Juan Creek after floods of February 10 and February 13, 1978.

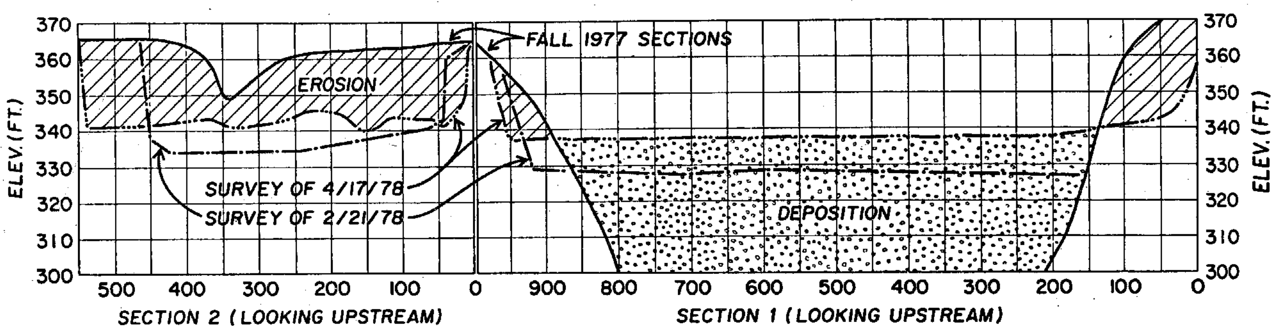


FIGURE 11 Erosion and deposition at upstream edge of gravel pit in San Juan Creek after flood of March 4, 1978.

the percentage by weight of the sediment that is finer than the size indicated in the table. For example, the first figure in the d_{50} column indicates that 50 percent of the sediment in sample 1 is finer than 6.0 mm--i.e., it has a median size of 6.0 mm. All the samples except sample 10 were taken from deposits in the mine excavation. As seen in the d_{50} column, the median size varies. The samples taken after the most recent and largest storm, on August 9, 1980, tend to be coarser than the others. This seems reasonable; however,

FIGURE 12 Bed of San Juan Creek approximately 1,000 ft upstream of park line, February 2, 1978.

because the samples were small this result may not be significant. It should be noted that the 3 percent sizes are from 0.14 to 0.32 mm, or in the fine to medium sand sizes. The percentage of the samples finer than 0.074 mm (not shown in Table 2) ranged from 0.2 percent to 0.9 percent. These data indicate that the deposits sampled contain very little silt and clay and that any of these fine sediments reaching the pit area were mostly carried through the pit. During the January floods and part of the February flows, when a substantial pond formed in the pit, as indicated by Figure 8, some of the silt and clay in transport was probably deposited.

Table 3 gives data on the floods of 1978 and the sediment deposited in the pit and eroded from the creek channels. The second and third columns show that the floods increased in intensity from January to March. Column 4 gives the volume of the deposits accumulated in the pit starting with the January flood, and column 5 gives the difference between the cumulative volumes of sediment deposited in the pit and eroded from the channels. The quantities shown in column 5 were assumed to be the cumulative sediment yield for the 1978 floods up to the dates indicated. The sediment volumes given in Table 3 were developed from surveys made after the January and the March floods. A survey was also made after the February floods but proved to be inadequate for determining the needed sediment volumes.

The mean sediment concentrations in the flow entering the pit and in the flow entering the degraded reaches of the creeks are given in columns 6 and 7

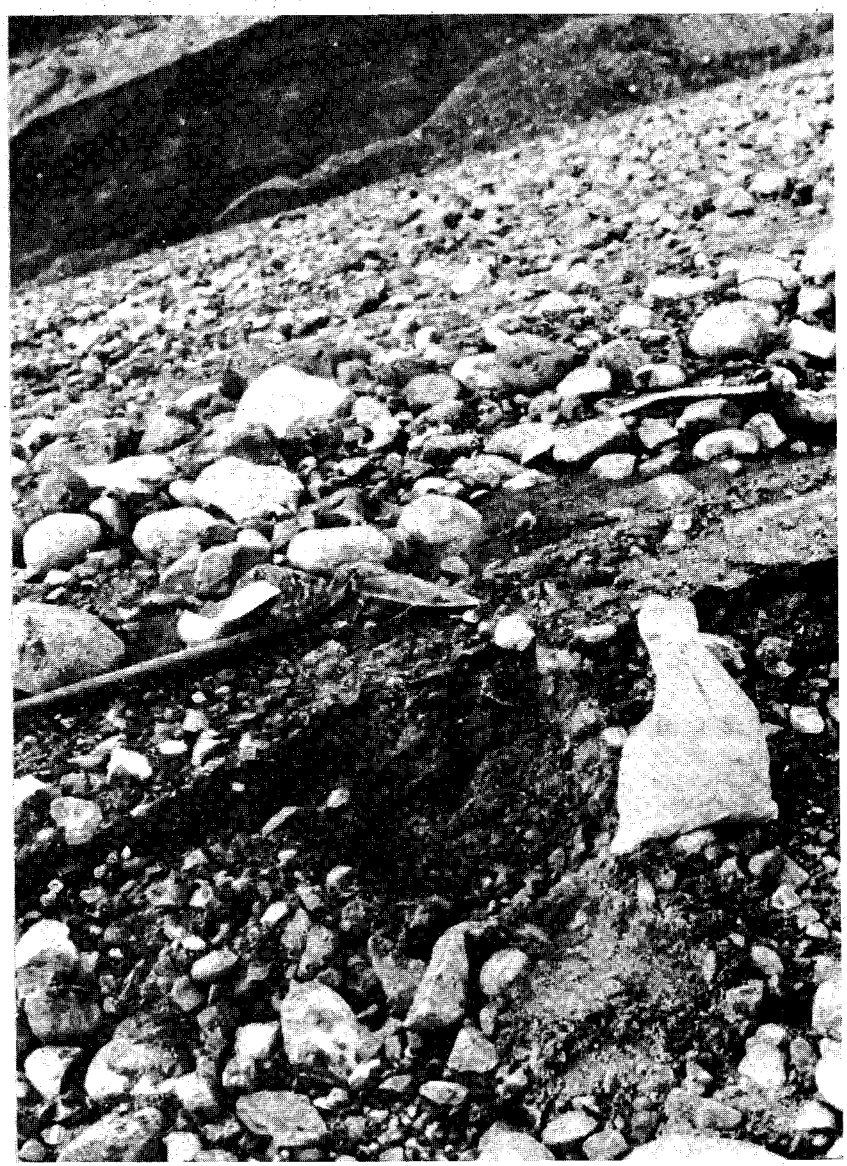


FIGURE 13 Surface of deposits in mine excavation 300 ft downstream of park line, February 6, 1978; site of sample 2 in Table 2.

of Table 3. In calculating these concentrations no account was taken of any of the fine sediments that may have been carried through the pit. It is surprising that the concentrations for the January floods were over four times the means for all of the 1978 floods despite the fact that the flow rates for January were much smaller than those for the subsequent floods. Two factors that tended to cause high sediment concentration in the January floods were that these floods followed several years of relatively low rainfall, so that large amounts of sediment probably accumulated in the drainage ways, and that the slopes in the degrading reaches near the pit were higher in January than in subsequent floods, causing unusually rapid erosion. The concept that

TABLE 2 Size Distribution of Bed Sediment of Creeks in and near Caspers Regional Park

No.	Date	Size (mm) d_3	d_{16}	d_{50}	d_{84}	d_{97}	Creek	Notes (ds = downstream from)
1	2/6/78	0.26	0.71	6.0	24	33	San Juan	100 ft ds park
2	2/6/78	0.32	0.75	7.8	32	36	San Juan	300 ft ds park
3	2/6/78	0.32	0.71	4.4	27	45	San Juan	640 ft ds park
4	4/11/78	0.16	0.65	12	118	140	San Juan	500 ft ds park
5	4/11/78	0.17	1.0	7.3	28	34	San Juan	1,000 ft ds park
6	8/9/80	0.14	1.2	32	54	85	Verdugo	Near mouth
7	8/9/80	0.34	2.8	20	78	85	San Juan	300 ft ds park[a]
8	8/9/80	0.14	0.25	0.67	4.3	12	San Juan	300 ft ds park[b]
9	8/9/80	0.25	17	25	56	62	San Juan	300 ft ds park[c]
10	8/9/80	0.25	14	45	78	88	Bell	At mouth

[a]200 ft from left bank of gravel pit.

[b]300 ft from left bank of gravel pit.

[c]500 ft from left bank of gravel pit.

sediment accumulates in a watershed in dry years to be washed away in storms is supported by the findings of Anderson, Coleman, and Zinke (1959). During a five-year study they found that the amount of sediment yielded from steep southerly slopes of the San Gabriel Mountains in southern California in the dry summer exceeded that eroded from these slopes during the rainy season.

The overall mean sediment concentration in the 1969 floods in San Juan Creek was approximately the same as in the 1978 floods, as indicated in the footnote of Table 3. The two floods in January and February 1969 had peak discharges of 16,000 and 19,000 cu ft/s, respectively, a combined runoff of 37,000 acre-ft, a sediment yield of 775,000 cu yd, and a mean concentration of 24 g/liter. The 1978 and 1969 floods were similar in that they followed several years of relatively low rainfall.

The last column of Table 3 gives an estimate of the relative sediment yield of each 1978 flood based on a sediment transport relation in which the sediment discharge rate, G_s, is proportional to $Q^{1.5}$, where Q equals the water discharge rate. This gave a negligible yield for January compared with a measured yield of approximately 20 percent of the total for 1978. This suggests that the actual transport relation for San Juan Creek does not resemble the simple power relations of water discharge found normally. It also indicates that there are difficulties in representing sediment discharge in streams such as San Juan Creek by standard power formulas.

TABLE 3 Sediment Deposited in Gravel Pit and Eroded from Watershed

1	2	3	4	5	6	7	8
Dates in 1978	Peak Discharge (cu ft/s)	Runoff (acre-ft)	Cumulative Sediment Yield (cu yd)		Sediment Concentration[b] (g/liter)		Estimated Percentage of Yield from Watershed[c]
			In Pit	Pit - Eros[a]	In Pit	Pit - Eros[a]	
1/15; 1/16	150	280					< 1
1/19; 1/20	165	300	143,000	55,000	280	110	< 1
2/10; 2/11	1810	2910					13
2/12; 2/13	1830	2580					12
3/4	11,600	8070	700,000	282,000	57	23	75

[a]The difference between the volume of sediment deposited in the pit and that eroded from channels is taken as the sediment yield from the watershed.

[b]Calculated average based on cumulative sediment yield and cumulative runoff through dates indicated.

[c]Estimate based on sediment discharge $\sim Q^{1.5}$. Floods of January-February 1969: sediment yield = 775,000 cu yd, runoff = 37,000 acre-ft, concentration = 24 g/liter.

One can ask what will be the future of the streams in the vicinity of the gravel pit. It is seen from Figure 8 that the slopes upstream of the pit went from a natural value of 1 percent in 1977 to over 3 percent in early February to 2.0 percent and then back to 1.6 percent in the spring of 1978. The slope of San Juan Creek, about 1,500 ft downstream of the park, is now (in 1980) less than 1 percent. Despite recovery in slope, the beds of the creeks in the park in the spring of 1978 were over 10 ft lower than in their undisturbed state, as shown by Figure 8. Further raising of the streams will require raising of the bed downstream. This level is now controlled by the channel shown in Figure 14, which is approximately 1,800 ft downstream of the park and is excavated in rock. To raise the level of this channel and that of the channels in the park will require aggradation of the bed downstream of the rock channel, which presently is inhibited by continuing gravel mining.

REFERENCES

Anderson, H. W., Coleman, G. B., and Zinke, P. J. (1959) Summer Slides and Winter Scour, Technical Paper No. 36, Pacific Southwest Forest and Range Experiment Station, Forest Service, U.S. Department of Agriculture, Berkeley.

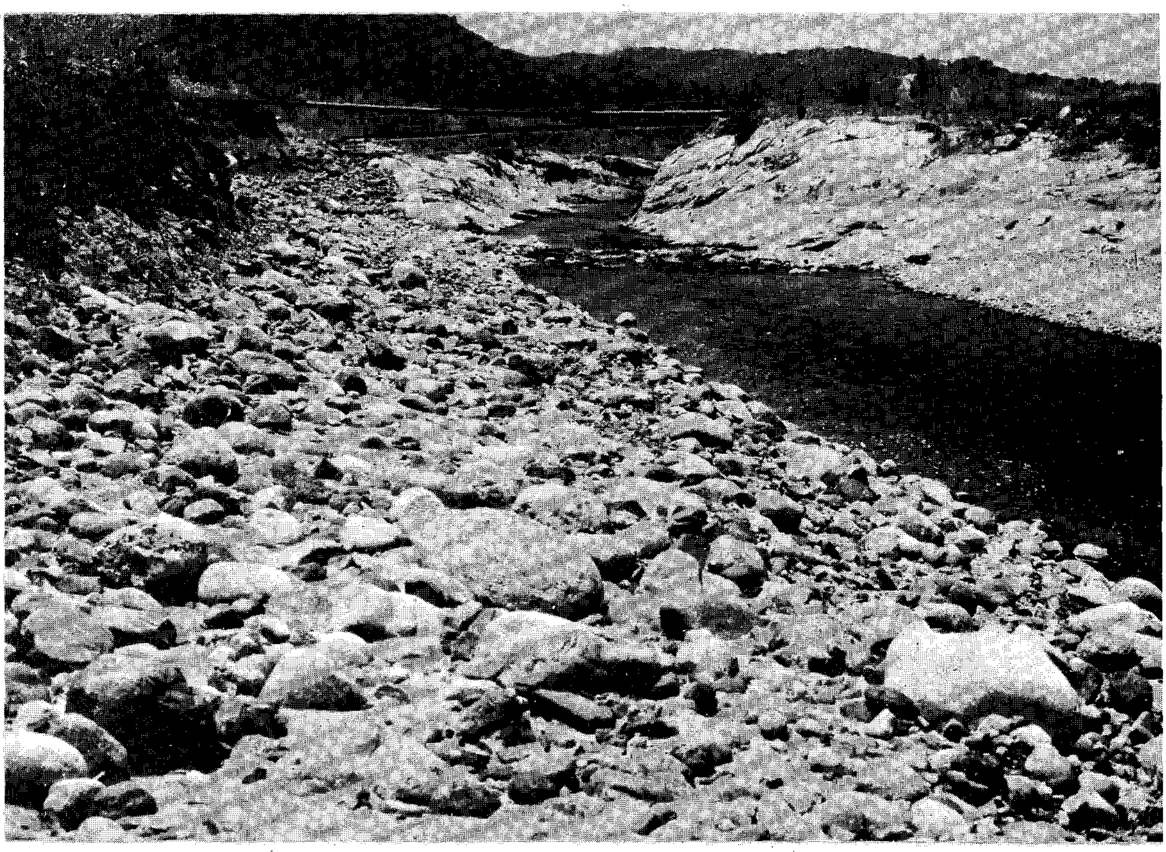


FIGURE 14 Downstream view of San Juan Creek in rock channel 1,800 ft downsteam of park, May 2, 1980.

Camp Dresser & McKee, Inc. (1978a) Preliminary Analysis of Flood Hazards Resulting from an Intermediate Regional Flood Upstream from Conrock Gravel Pit, San Juan Creek, Orange County, Calif., Camp Dresser & McKee, Newport Beach, California, January 6.

Camp Dresser & McKee, Inc. (1978b) Supplemental Report on Erosion and Deposition Processes in and above Conrock Gravel Pit, San Juan Creek, Orange County, Calif., Camp Dresser & McKee, Newport Beach, California, March 20.

Converse Ward Davis Dixon (1979) Inter-relation of Landslides, Fire, Erosion and Sedimentation in San Juan Creek Drainage, Southern California, Guidebook to Selected Geological Coastal Areas of Southern Orange and Northern San Diego Counties of Southern California, South Coast Geological Society, Anaheim, California, pp. 167-184.

Creager, Justin, and Hinds (1947) Engineering for Dams, Vol. I, John Wiley & Sons, New York, p. 125.

Hollingshead, A. B. (1971) Sediment transport measurement in gravel rivers, Journal of Hydraulics Division, ASCE 97 (November 1971):1817-1834.

Mostafa, M. Gamal (1978) Erosion of San Juan Creek in the Vicinity of Caspers Regional Park, Orange County Environmental Management Agency, Anaheim, California, March 10.

SOUTHERN CALIFORNIA LANDSLIDES OF 1978 AND 1980

by James E. Slosson and James P. Krohn

The high-intensity rainfalls of 1978 and 1980 brought devastation in the form of landslides, debris flows, and mudflows. Losses from the storm-related phenomena within the City of Los Angeles alone amounted in 1978 to an estimated $50 million for private property and approximately the same amount for public property. In 1980, with inflation greatly affecting the value of the dollar, the losses are estimated to have reached $70 million for private property and approximately the same amount for public property. Severe damage and losses from landslides, debris flows, and mudflows are not uncommon to southern California (or the United States, for that matter), with the high-intensity or abnormally high rainfalls of 1938, 1952, 1958, 1962, and 1969 also damaging or destroying large numbers of structures.

Deaths have been recorded from these landslide-type failures, but at a lower rate than from other natural hazards, considering the equivalent dollar loss. Available records indicate that these storms have caused 6 to 12 deaths per event. Mudflows and debris flows have been the killers, with only one death directly attributed to a bedding plane or arcuate landslide and one to the failure of a retaining wall.

Storm damage data collected by the Department of Building and Safety of the City of Los Angeles for the storms of 1969, 1978, and 1980 substantiated the opinion that proper use of science and technology, coupled with realistic and enforceable codes, can reduce losses from natural hazards. The City of Los Angeles was the first jurisdiction to adopt a grading ordinance; this followed the multimillion dollar losses from the storm of 1952. This ordinance was modified again after the 1958 storm losses. Finally, a modern, effective code was developed and put into effect in 1963 following the storm of 1962.

Damage and loss data collected and computerized by the City of Los Angeles from the 1969, 1978, and 1980 storms lucidly illustrates that over 90 percent of the losses were associated with the pre-1963 properties (and structures). The remaining 10 percent of the losses appear to be related primarily to natural and post-1963 engineered fill slopes. The number of sites developed before and after 1963 are both in the 30,000+ range.

James E. Slosson and James P. Krohn are Engineering Geologists with Slosson and Associates in Van Nuys, California.

Damage from debris flows and mudflows appears to be increasing in magnitude and is caused, in part, by the increased construction of homes at the base of natural slopes or partial natural slopes associated with older subdivisions. Most severely hit appear to be those sites or lots that were a part of pre-1963 or even pre-1952 subdivisions but were not built upon until recent years. This general relationship is especially true for Topanga Canyon, Mandeville Canyon, Stone Canyon, and portions of Sherman Oaks, Encino, Tarzana, and Woodland Hills. The potential for mudflow and debris flow hazard is easily recognized, but few consultants will acknowledge evidence unless required by code.

INTRODUCTION

The high-intensity rainfalls of 1978 and 1980 brought devastation in the form of landslides, debris flows, and mudflows (Figures 1 and 2). Losses from the storm-related landslide phenomena within the City of Los Angeles amounted in 1978 alone to $50 million for private property and an assumed equal amount for public property. The 1980 losses for the City of Los Angeles have been estimated at $75 million for private property, with probably an equal loss for public properties and facilities. The estimated total loss within the six southern counties affected from landslides, debris flows, mudflows, etc. in 1980 approximates $500 million.

As determined by our studies and those of others (e.g., Fleming and Taylor, 1980), record keeping by most local, state, and federal agencies regarding landslide designations, damages, and losses ranges from less than adequate to none. The City of Los Angeles is an exception to this generalization, as it keeps excellent records and subsequently makes them available for research.

The high-intensity rainfalls of 1978 and 1980 and their associated floods and landslides are not an isolated storm hazard for southern California. Similar storms with associated floods and landslides were recorded in 1952, 1958, 1962, and 1969. Earlier years witnessed similar rainfall and flood relationships, but no records are available to determine the extent of landslide losses. Aerial photos taken during this period between 1927 and 1952 indicate that landslide activity did occur during these years of high-intensity rainfall. However, (1) there were fewer homes to be affected, (2) there was no television to highlight the problem, (3) governmental agencies and the courts were assuming landslides and the associated losses were an "act of God" and were not recognizable or preventable, (4) most of the geologists during the pre-1952 era had not been trained to recognize and/or mitigate landslides, (5) very few universities offered courses in engineering geology and/or soil mechanics prior to the 1950s, and (6) civil engineers did not appreciate the involvement of geologists until the early 1960s.

Records prior to the first use of aerial photographs in southern California (in 1927) were limited to newspaper articles and a few case histories. However, even with the poor records there is still indication of disastrous landslides (including mudflows and debris flows) during periods of high-intensity rainfall. Figure 3, a bar graph of the annual rainfall for Los

FIGURE 1 The Bluebird Canyon landslide (in October 1978) damaged or affected more than 50 homes in a hillside development in Laguna Beach.

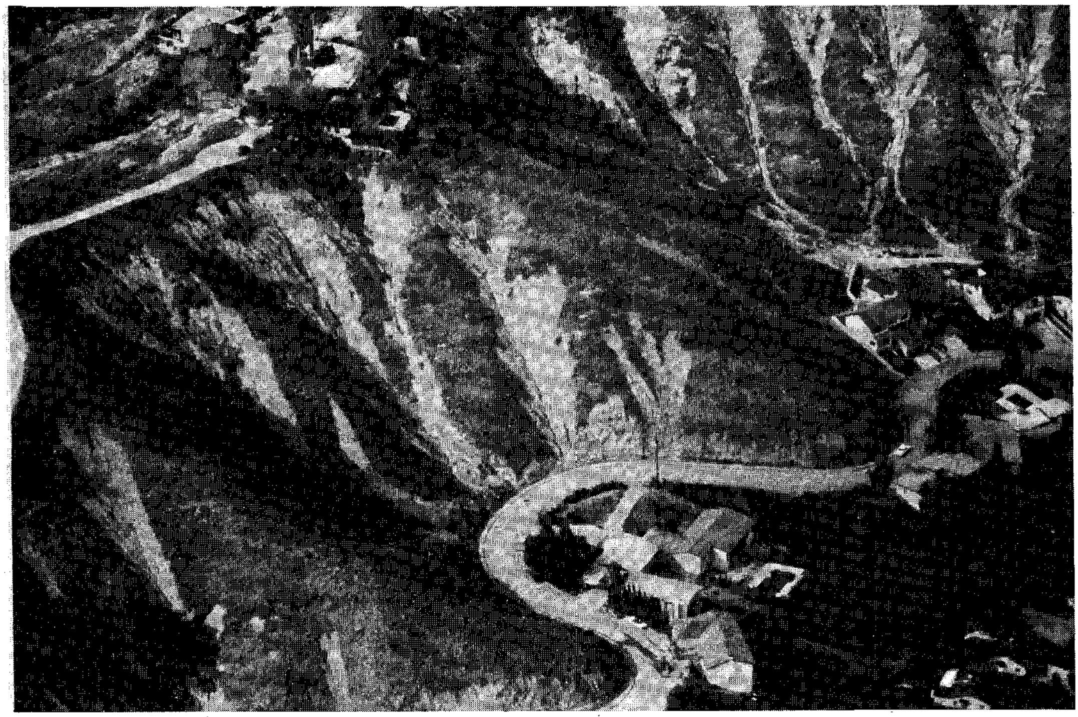


FIGURE 2 Debris flows and mudflows on natural slopes of a pre-1952 subdivision from a 1980 storm in Woodland Hills, California.

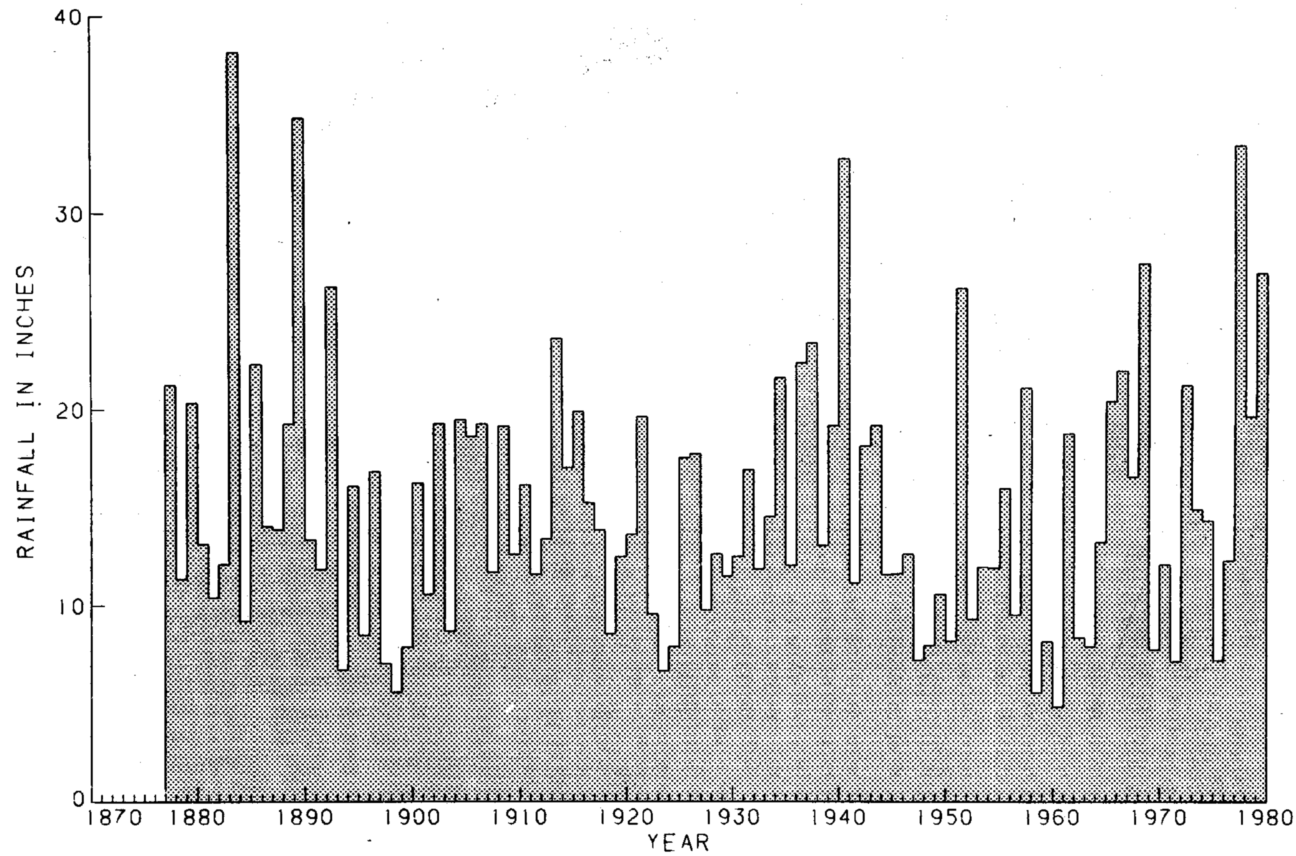


FIGURE 3 Annual rainfall in Los Angeles, 1877-1980.

Angeles (in the center of the city), depicts the rainfall pattern for the past 103 years. The average annual rainfall for Los Angeles is 15.06 in. Flood and landslide damage is generally associated with those years when the rainfall has been greater than average by about 30 percent or more (i.e., over 20 in.). The year 1884 may have been the most severe with respect to the area of flooding and landslide activity. News articles and other references describe widespread flooding extending from southern California to central Arizona. Some references suggest that essentially all bridges were destroyed between Los Angeles and Tucson. Tentative age dating of some of the larger previous landslides of southern California indicates an approximate age of 100 years, which would be in agreement with the assumed damage attributed to the storms of 1884.

A review of the rainfall bar graph indicates that there have been years when the rainfall was above 20 in. (130 percent of average) without the damaging landslides and flow that occurred in 1978 and 1980. This caused speculation on the part of the authors and others as to what other parameters were involved in the genesis of landslides. Review of rainfall data indicated that not only has there been above average rainfall but there has been a pattern of at least five or more days of high-intensity rainfall during which at least 7 in. of rainfall has been recorded. The most disastrous years appear to be those when the greatest rainfall in a 24-hour period occurs after five days of rain amounting to over 7 in.

In 1980 the rainstorm started on February 8. The sequence of five days of continuous rain and 7 in. of precipitation had occurred by February 14. Slope failures were beginning to develop by February 15 and then very-high-intensity rainfall occurred on February 16. Examples of short-period high-intensity rainfall were available from the recording stations at Encino Reservoir, where 8 in. of rain fell between noon and 6:00 p.m., and at Sepulveda Dam, where 7 in. fell during the same six-hour interval.

Records and personal observations in the field on February 16 and 17 show that the mountains and slopes literally fell apart on those two days. One of us (Slosson) was in the Pacific Palisades area from 11:00 a.m. to 1:00 p.m.; at the Pacific Coast Highway-Malibu from 1:00 p.m. to 3:00 p.m.; at Topanga Canyon, Saddle Peak Road, Piuma Road, Stunt Road, and Mulholland Drive from 3:00 p.m. to 5:00 p.m.; at Woodland Hills, Encino, and Sherman Oaks from 5:00 p.m. to 6:00 p.m.; and in Calabasas, Agoura, the Santa Monica Mountains between Topanga Canyon and Kanan Drive Road, and Malibu on the seventeenth. During these two days landslides, debris flows, mudflows, and aggressive erosion activity were witnessed and photographed. The greatest frequency of landslides appeared, from personal observation, to occur between 11:00 a.m. and 6:00 p.m. on Saturday (the sixteenth) and between 3:00 p.m. and 7:00 p.m. on Sunday (the seventeenth).

Similar but less extensive observations made during the high-intensity rainfall and associated landslide activity in 1952, 1958, 1969, 1978, and 1980 indicated a similar sequence of events: five or more days of rainfall amounting to at least 7 in. Superimposed over these were short periods of high-intensity rainfall.

Continued rainfall of five days or more causes saturation of not only the soil but other loose surficial materials (such as alluvium, colluvium/-slopewash, nonengineered fill) and underlying rock materials. This saturation causes the following changes in the physical characteristics of these materials.

1. Increase in bulk weight as water fills the poor space
2. Reduction in strength as (a) the availability of water between the particles reduces friction and (b) water separates clay and silt particles, reducing cohesion
3. A general increase in pore water pressure
4. Solution (the removal of natural cementing materials)

Statistics collected by the Los Angeles City Department of Building and Safety over the past 30 years have provided a good data base from which to estimate the dollar loss from landslides, mudflows, etc. These estimates are listed in Table 1. During this 30-year period the number of structures increased by approximately 50,000 (from 15,000-20,000 to approximately 70,000) while the values of homes increased by a factor of almost ten. Construction

TABLE 1 Losses from Landslide Activity (adjusted to inflation)

Year	Loss (millions of dollars)	Sample Cost of Hillside Tract House (dollars)
1952	7.5	20,000
1958	7.5	35,000
1962	12.0	50,000
1969	10.0	85,000
1978	50.0	125,000
1980	70.0	175,000

costs during this era increased from about $7.50/sq ft to $35.00-$55.00 to $60.00/sq ft, and lot costs increased from $2,500-$5,000 to $35,000-$60,000. Thus inflation may have a very important control on the dollar losses. The skyrocketing increase in the dollar losses can be misleading. Equating the losses to 1952 dollar equivalents, the estimated dollar loss per home (i.e., considering the increase in the number of houses) has decreased.

Statistics derived from the City of Los Angeles Department of Building and Safety for the high-intensity rainfall of 1969 and 1978 have been scanned to determine the relationship between losses to storms and the development and enforcement of grading codes. Table 2 shows previously developed statistics for 1969, and Table 3 shows statistics for the 1978 storm. The excellent methodology for collecting and computerizing landslide loss was not established by the City of Los Angeles until 1969. Thus statistics of similar quality are not available for earlier storms in 1952, 1958, and 1962. Computerized data from the 1980 storm were not completed by the date of this conference.

It is obvious from both the 1969 and 1978 computerized storm data that the codes developed and enforced by the City of Los Angeles have been extremely effective in mitigation. Table 2 (the 1969 storm) shows that damages related to landslides, mudflows, etc. have been reduced by 97 percent. Table 3 (the 1978 storm) shows an effective reduction of approximately 95 percent. It is estimated that the 1980 storm statistics will show a loss reduction (or mitigation factor) of at least 92 to 95 percent.

This slight decrease in loss reduction (mitigation factor) from 97 to 92 to 95 percent we believe to be attributed to the following.

1. Relaxed enforcement, with the axiom being "the further from the last storm (in years), the less rigorous the enforcement and concern"
2. Loss of well-trained grading inspectors through promotion or retirement
3. Relaxation of, or elimination of, adequate training programs for grading inspectors

TABLE 2 Damage to Building Sites Under Different Building Codes (1969 storms)

Pre-1952	1952-1963	1963 to Present
No grading code, no soils engineering, no engineering geology.	Semiadequate grading code, soils engineering required, very limited geology, but no status and no responsibility.	New modern grading codes; soils engineering and engineering geology required during design; soils engineering and engineering geology required during construction; design engineer, soils engineer, and engineering geologist all assume legal responsibility.
Approximately 10,000 sites constructed.	Approximately 27,000 sites constructed.	Approximately 11,000 sites constructed.
Approximately \$3,300,000 damage.	Approximately \$2,767,000 damage.	Approximately \$182,400 damage.[a]
Approximately 1,040 sites damaged.	Approximately 350 sites damaged.	Approximately 17 sites damaged.
An average of \$330 per site for the total number produced: $\frac{\$3,000,000}{10,000 \text{ sites}}$	An average of \$100 per site for the total number produced: $\frac{\$2,767,000}{27,000 \text{ sites}}$	An average of \$7.00 per site for the total number produced: $\frac{\$80,000}{11,000 \text{ sites}}$
Predictable failure percentage: 10.4 percent $\frac{1,040 \text{ damaged}}{10,000 \text{ total sites}}$	Predictable failure percentage: 1.3 percent $\frac{350 \text{ damaged}}{37,000 \text{ total sites}}$	Predictable failure percentage: 0.15 percent $\frac{17 \text{ damaged}}{11,000 \text{ total sites}}$

Note: The storms of 1962, 1957-58, 1962, 1965, and 1969 all produced similar total losses associated with similar destructive storms.

[a] Over \$100,000 of the \$182,000 was incurred on projects where grading was in operation and no residences were involved; thus less than \$80,000 occurred on sites constructed since 1963.

Source: Slosson (1969).

4. Loss of soil engineers from the staff
5. Change in the membership of both the building safety commission and the city council--generally with people who have not gone through the experience of a disastrous winter being replaced

Tables 2 and 3 show that effective codes can reduce losses. In 1952 the City of Los Angeles recognized that high-intensity rainfall may cause landslides and associated monetary losses. In an attempt to reduce these losses, the Department of Building and Safety proposed the first grading code

TABLE 3 Slope Failures in the City of Los Angeles (1978 storms)

Description	Number of Sites	Number of Failures	Percent Failure	Dollar Value (millions)
Pre-1963 (before modern code)	37,000	2,790	7.5	40-49
Post-1963 (modern code)	30,000	210	0.7	1-2

Note: The categories of failure are (1) soil slippage and erosion (28 percent); (2) mudflow and debris flow (30 percent); (3) slump/arcuate landslides, pre-1963 and natural slopes (22 percent); (4) reactivation of old failures, pre-1963 (8 percent); (5) new bedrock landslides, pre-1963 (5 percent); (6) shallow fill slope and some natural slope failure, post-1963 (7 percent, with the modern code promulgated in April 1963).

Source: Slosson and Krohn (1979).

that was ever used. Many believed that this code, which was minimal in nature, would solve the problem. This code required that the soil engineer or design civil engineer determine the geologic hazards. The concept that one discipline could decide on the need for expertise from another discipline was proven to be in error, as the geologic problems were overlooked by those who had neither the training nor the experience to determine whether a hazard existed or not. Another factor was that the developers were usually more interested in keeping the "up front" costs to a minimum than in reducing landslide losses.

Unfortunately (or fortunately, for the more recent buyers of new homes), the 1958 storm proved the 1952 code to be inadequate and ineffective. Following the 1958 storm the City of Los Angeles established an ad hoc committee composed of engineers and geologists to assist in the preparation of a revised and upgraded code. This code required that geologic reports be submitted, but did not specify by whom. In addition, the geologist was seldom involved in the development of design criteria and/or concepts and was seldom required to inspect the tract during grading. As a result, the geologic input and involvement were of very limited value for disaster mitigation.

The high-intensity rain of the winter of 1962 again brought devastation to California, and particularly to the hillside areas of Los Angeles. The mayor, the city council, the Department of Building and Safety, the professions of civil engineering, engineering geology, and soil engineering, homeowners organizations, and the news media all asked for a stop to losses and damage from landslides; some asked for a moratorium on all hillside construction.

Cooperative efforts between the professions of engineering and geology, working with the City of Los Angeles, resulted in a new and modern grading code that became effective in April 1963. Statistics from the 1969 and 1978 storms show that this code has reduced losses by 95 to 97 percent. The most important additions to the 1963 code were (1) the requirement that each professional (the design [civil] engineer, the soil [civil] engineer, and the engineering geologist) is required to provide inspection during the grading of tracts; (2) the requirement that these professionals perform adequate professional analysis prior to issuance of a grading permit; (3) the requirement that these professionals certify and sign (with the appropriate license/registration numbers noted) the grading plan and the "as-built" grading plan; (4) the requirement that the project be certified as being completed in accordance with the plans and good professional standards; and (5) the requirement that the engineering geologist and soil engineer comply with standard procedures established by the City of Los Angeles for exploration and slope stability analysis.

Appendix A contains excerpts from the 1963 codes for the City of Los Angeles. Appendix B consists of similar provisions in the grading codes for Ventura County, which have produced very good results in reducing losses from landslides. In addition to the codes, the Board of Registration for Geologists and Geophysicists in California has submitted "Recommended Guidelines for the Preparation of Engineering Geologic Reports" to all geologists registered in California. The guidelines, which were prepared by the California Division of Mines and Geology, appear in Appendix C.

Tables 2 and 3 show that not only has there been a reduction in total dollar (or monetary) losses and the number of sites affected but also that the loss per site for those damaged has been reduced. A review of the data base indicates that there have been no landslide failures (slump, arcuate and planar, or bedding plane) on tracts developed after 1963. The post-1963 losses have been related to engineered fill failures (not geologic failures) and mudflows or debris flows from natural or engineered fill slopes (Figures 4, 5, and 6).

In 1978 the predominant problem noted in the post-1963 tracts was shallow slump or flow failures in the outer face (slope face) of engineered fills (Figure 1). These failures usually occurred within the outer 2 to 5 ft of the fill slope. This problem should be addressed by the engineering profession, or a warranty similar to a product warranty (such as a one-year warranty on an appliance) should be provided to ensure repair--possibly it should be a 10-year warranty. This warranty could be financed by the buyer, such as is done for some major appliances, or by the seller (distributor). In lieu of a warranty, insurance should be made available, just as flood insurance was made available when it became apparent that homeowners needed assistance.

In 1980 the number of mudflows and debris flows from natural slopes increased. Possible reasons for the increase in this damage factor include the following.

1. Construction on isolated lots in the older tracts (almost all dated from before 1963) where a potential for mudslides or debris flows existed (Figure 7).

FIGURE 4 Rear yard and house of a pre-1963 subdivision inundated by mudflow off a natural slope during a 1980 storm in Tarzana, California.

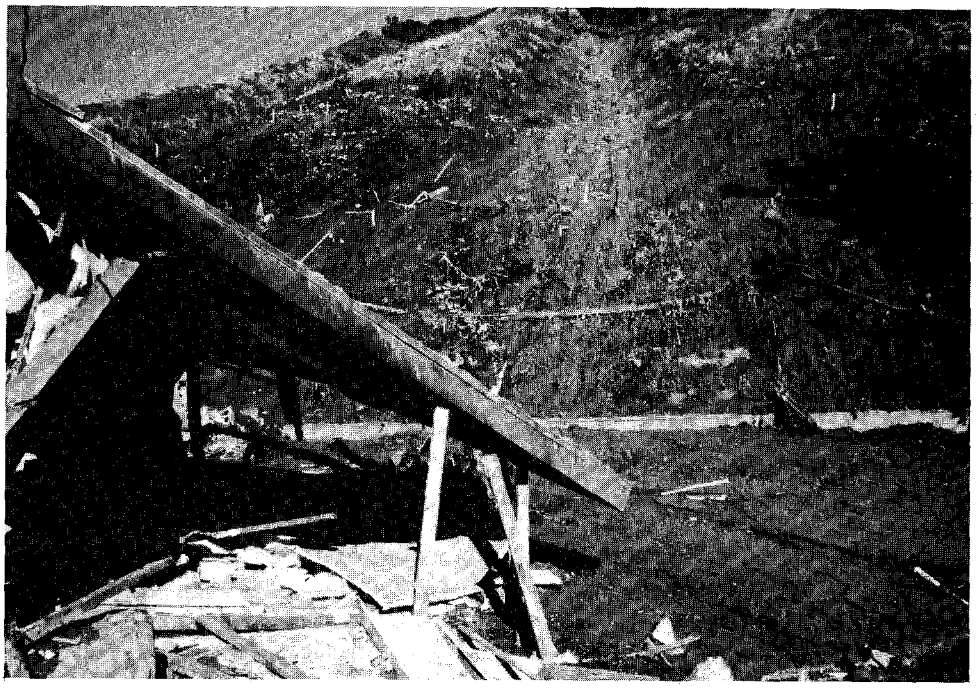


FIGURE 5 House of a pre-1963 subdivision destroyed by mudflow from a natural slope failure in Woodland Hills, California. (Note: Two debris fences from repairs made after a 1978 storm, upslope from the house, proved to be totally ineffective against the mudflow.)

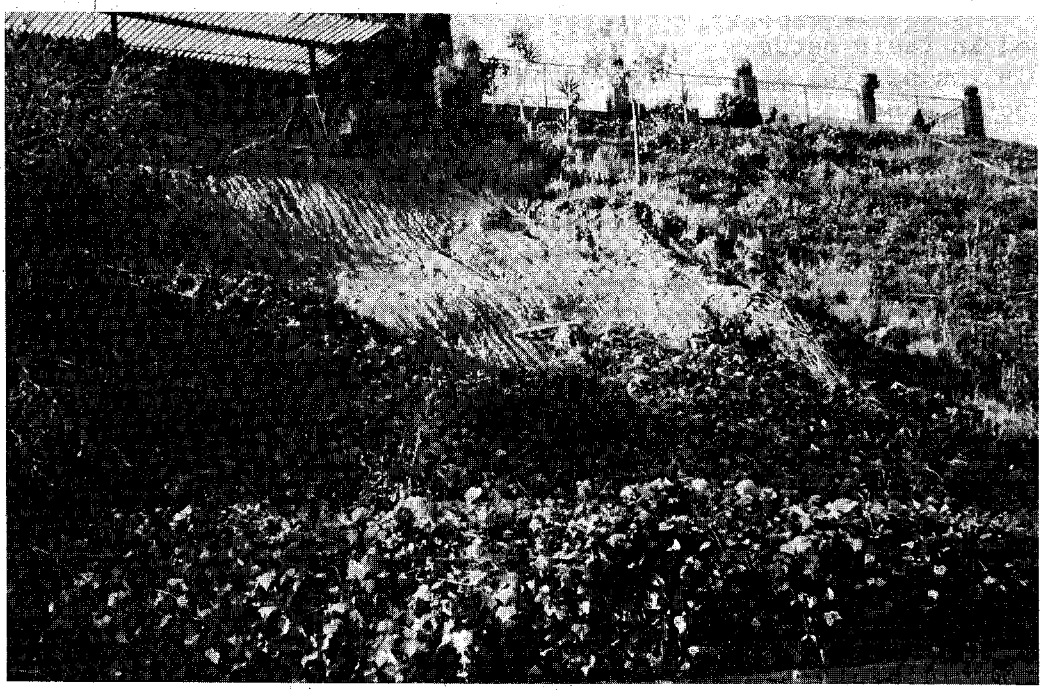


FIGURE 6 Surficial failure in compacted engineered fill slope from a 1980 storm in La Habra, California.

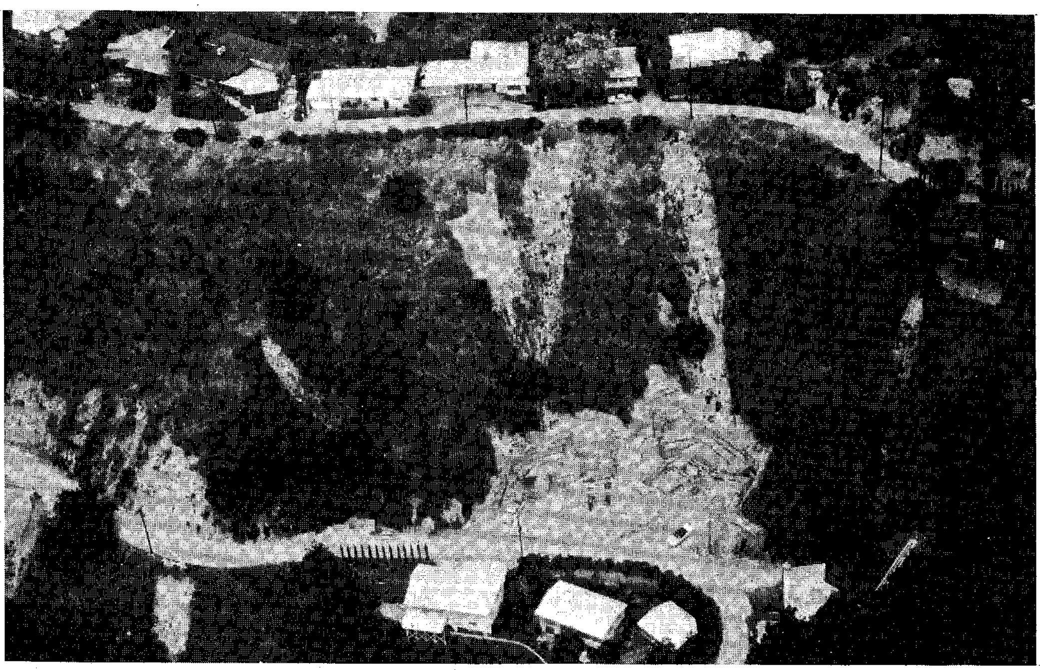


FIGURE 7 New construction occurring downslope from 1980 storm debris flow and mudflow scars in Studio City, California.

2. The requirement by some regulatory provisions that hilltops be retained in their natural condition and/or that homes be constructed, wherever possible, on natural slopes. This inadvertently allows houses to be constructed where a potential for mudflows or debris flows exists.

3. Failure by some geologists, soil engineers, and governmental officials to recognize the hazard.

It is hoped that future code revisions will address both of these problems: (1) failure on the outer face of fill slopes and (2) mudflows or debris flows from natural slopes. We believe that mitigation is possible in both cases. New technology or construction practices may be necessary to correct the problem of fill slope failures in conjunction with better methodology and guidelines for recognizing the potential for mudflows and debris flows. For instance, R. H. Campbell (1975) wrote an excellent paper on why, where, and when mudflows and debris flows have occurred and should occur. Technical reports of this type should be required reading for all geologists and civil engineers associated with hillside construction.

We have continually referred to the City of Los Angeles because (1) the City of Los Angeles has the best and most complete records; (2) the City of Los Angeles has the best and most effective grading code; (3) the City of Los Angeles provides good review and inspection; and (4) the reduction in losses for post-1963 tracts within the City of Los Angeles has been approximately 95 percent, which from all available data is the most effective loss reduction known. Other jurisdictions, such as the City of Thousand Oaks and the counties of Los Angeles, Orange, and Ventura, have had noticeable improvements while others have had little to none. All cities and counties are required by California statutes to use Chapter 70 of the Uniform Building Code (UBC) where there is hillside grading. Some are using Chapter 70, some have adopted it but are not enforcing it, and others have not even bothered to adopt it.

The news media and politicians frequently refer to the losses in Los Angeles and southern California with the suggestion that landslides, mudflows, and debris flows only occur here. This concept is false, as losses per capita or per home are higher in the San Francisco area than in Los Angeles (e.g., the losses in 1969 in the City of Los Angeles and County of Los Angeles were approximately $20 million, whereas an equivalent number of homes in the San Francisco Bay area received approximately $25 to $35 million damages). Fleming and Taylor (1980) suggest that very few jurisdictions keep good records, with some placing landslide losses under other categories. The estimated losses in the 48 conterminous states per year, even with incomplete records, total $1 billion per year.

Lessons learned from storm-related landslide damage within the southern California area (and especially the City of Los Angeles) can be of great value to the remaining portions of the country. Acting on such lessons (e.g., implementing grading codes) could be instrumental in reducing the estimated $1 billion landslide-related losses that occur annually in the United States.

Data suggest further that landslides may cause greater domestic or social strife (such as divorce, separation, and suicide) than any other natural hazard or disaster. Data collection and research should be encouraged. Earthquake-related domestic, social, or psychological problems have received much attention and funding. It appears that very little has been earmarked to study the effects of landslides and floods and the long-range consequences of these events to the families involved. This appears to be an important lacuna, since the dollar losses appear to be about the same.

REFERENCES

Campbell, R. H. (1975) Soil Slips, Debris Flows, and Rainstorms in the Santa Monica Mountains and Vicinity, Southern California, USGS Professional Paper 851, U.S. Geological Survey, Reston, Virginia.

Fleming, R. W., Varnes, D. J., and Schuster, R. L. (1979) "Landslide Hazards and Their Reduction," Journal of American Institute of Planners 45(No. 4): 428-439.

Fleming, R. W., and Taylor, F. A. (1980) Estimating the Costs of Landslide Damage in the United States, USGS Circular 832, U.S. Geological Survey, Reston, Virginia.

Kockelman, W. J. (1980) Tools to Avoid Landslide Hazards and Reduce Damage, USGS Open-File Report 80-487, U.S. Geological Survey, Menlo Park, California.

Krohn, J. P., and Slosson, J. E. (1976) "Landslide Potential in the United States," California Geology 29(No. 10, October 1976):224-231.

Leighton, F. B. (1976) Urban Landslides--Targets for Land-use Planning in California, Special Paper 174, Geological Society of America, Boulder, Colorado, pp. 37-60.

Los Angeles Times (1980) Part 1, p. 1, June 29.

Nilsen, T. H., Taylor, F. A., and Brabb, E. E. (1976) Recent Landslides--An Estimate of Economic Losses and Correlation with Slope, Rainfall, and Ancient Landslide Deposits, USGS Bulletin 1398, U.S. Geological Survey, Reston, Virginia.

Schuster, R. L., and Krizek, R. J. (1978) Landslides Analysis and Control, Special Report 176, Transportation Research Board, National Academy of Sciences, Washington, D.C.

Slosson, J. E. (1969) The Role of Engineering Geology in Urban Planning, Colorado Geological Survey Special Paper 1, Colorado Geological Survey, Denver, pp. 8-15.

Slosson, J. E., and Krohn, J. P. (1979) "AEG Building Code Review--Mudflow/Debris Flow Damage, February 1978 Storm--Los Angeles Area," California Geology 32(No. 1, January 1979):8-11.

Stokes, A. P., and Cilweck, B. A. (1974) "Geology and Land Development in Ventura County, California," California Geology 27(No. 11, November 1974):243-251.

Weber, F. H., Jr., and Treiman, J. A. (1973) "Slope Instability and Debris Flows," California Geology 32(No. 1, January 1979):3-4.

Wiggins, J. H. (1978) Building Losses from Natural Hazards: Yesterday, Today and Tomorrow, NSF Grant No. ENV-77-08425, Washington, D.C.

APPENDIX A: EXCERPTS FROM THE CITY OF LOS ANGELES OFFICIAL GRADING REGULATIONS

DEPARTMENT OF BUILDING AND SAFETY, CITY OF LOS ANGELES BUILDING BUREAU, RULE OF GENERAL APPLICATION--RGA 4-67

Subject: Rules and Regulations for Supervision on Hillside Tract Grading--RR 23352

The permittee shall employ a registered civil engineer or land surveyor to prepare the design of grading plans for all hillside grading. The design civil engineer or land surveyor shall prepare his design in accordance with good planning practice, applicable Codes and to the restrictions imposed thereon as determined by detailed studies of the site and materials to be graded. These studies shall be performed by a soils engineer and an engineering geologist approved by the City of Los Angeles and shall be submitted prior to issuance of permits. The design civil engineer or land surveyor shall furnish sufficient supervision during construction to obtain compliance with the plans, as approved.

The permittee shall employ a soils engineer and an engineering geologist prior to planning the tract, whose duties shall be: to work closely with the design civil engineer or land surveyor, to examine surface and subsurface conditions in accordance with the Rule of General Application dealing with "Subsurface Exploratory Work" and to submit reports thereon. These reports, in conjunction with the Ordinance, shall form the basis for the design of the grading project. These reports shall be based upon a detailed topographic base map of the area to be graded and shall include specific conclusions and recommendations for avoidance or correction of all known existing or anticipated geologic hazards on or affecting the site or contiguous property.

The soils engineer, in addition to his pre-grading exploratory work, shall provide inspection during the placement of all compacted fill in accordance with the requirement of the Ordinance, the approved plans and good engineering practice. In addition, he shall follow the progress of the job sufficiently close to determine that the recommendations of his pre-grading report are followed. If conditions which require modification of plans are encountered during grading, he shall submit a report of his findings and recommendations for change of plans to the permittee and the design civil engineer, the engineering geologist and the Department.

The engineering geologist, in addition to his pre-grading exploratory work, shall provide inspection during the actual grading process at least as often as determined to be appropriate by the Department or Board, with periodic in-grading inspection reports submitted at intervals determined by the Department. Such grading inspection by the engineering geologist is to determine that the conditions of his pre-grading report are as anticipated. If conditions which require modification of the plans are encountered during grading, he shall submit a report of his findings and recommendations to the committee, design civil engineer or land surveyor, soils engineer and the Department.

The soils engineer, at the completion of grading, shall submit a certified report of compaction tests for all fill located within the limits of the tract and/or offsite grading areas. The soils engineer's final report shall also include: a statement that all sub-drains were installed, his professional opinion of the suitability of the fill placement area and the ability of the natural materials to support the compacted fill without excessive settlement of the fill or potential damage to structures erected thereon, a statement to the effect that he has inspected all cuts and fills and that in his opinion they meet the design requirements. The report shall be referenced to a dated as-graded plan prepared by the design civil engineer or land surveyor.

The engineering geologist at the completion of grading shall submit a final geologic report stating that: he had maintained the required in-grading inspection, the recommendations of his pre-grading report(s) have been followed, that in his professional opinion all known adverse geologic conditions have been corrected or provided for, future adverse geologic conditions are not anticipated, and all lots or sites are geologically suitable and safe for construction. The report shall include the geologist's certification that he has inspected all cut slopes and sidehill fill placement areas prior to placement of fill. He shall also certify that all sub-drain placement areas were inspected prior to installation of the sub-drains. The report shall be referenced to a dated as-graded plan prepared by the design civil engineer or land surveyor.

Upon completion of grading, the civil engineer or land surveyor responsible for the design shall submit an as-graded plan to the Department for approval of all work covered by the grading permit(s) and shall include the following:

1. The plan shall be at a 1 inch = 40 feet scale and shall show the locations of streets, pads, slopes, structures, pertinent elevations, original contours and finished elevations, other pertinent information required to show the as-graded condition, and shall be dated.

2. The plan shall bear the signature of the design civil engineer or land surveyor which shall certify that he has inspected the site, reviewed the plans and that the work shown and completed is in accordance with his design.

If, for any reason any of the three professional persons is terminated during the progress of the grading work, he and the committee shall immediately notify the Department in writing. Such termination may result in temporary delays in the grading operations until satisfactory arrangements are made to assure the Department that competent professional supervision is provided. When one or all three of the professionals of record are terminated, the new professional(s) shall submit to the Department a letter of certification that the previous professional's designs, reports and recommendations have been reviewed, all provisions of the Board or Department required as conditions of the grading permit will be complied with during the course of the work, and he or they shall review the detailed 40-scale grading plans and thus assume his or their responsibility as herein specified for all future grading on the project. The letters shall be referenced to the approved grading plans prepared by the design civil engineer or land surveyor.

The certification submitted by the civil engineer or land surveyor shall pertain to the tract as built. The certification shall apply to the angle of stability of cut and fill slopes, compaction requirements, drainage provisions, and in general, all safety features incorporated in a well-graded hillside job. Engineers and geologists employed for the development shall not be deemed to be responsible for the work if alteration work not under their control is undertaken after the grading certificate has been issued.

RULE OF GENERAL APPLICATION--RGA 5-67

Subject: Rules and Regulations for Hillside Exploratory Work--RR 23353

The following rules and regulations shall apply on required hillside surface and subsurface exploratory work:

Surface and subsurface exploratory work shall be performed by a soils engineer and an engineering geologist approved by the City of Los Angeles on all hillside grading work, except wherein waived by the Department Staff or Board. Such exploratory work shall be performed for the purpose of obtaining detailed information on which the soils engineer and the engineering geologist shall base recommendations for a grading project. The work shall be based upon a detailed, accurate topographic base map prepared by the registered civil engineer or land surveyor. The map shall be of suitable scale, and shall cover the area to be graded, as well as adjacent areas which may be affected by the grading. The map shall include the existing and proposed contours, locations of streets, pads, slopes, structures, and pertinent elevations.

The engineering geologist's and soils engineer's exploratory work shall be conducted at locations considered most likely to reveal any subsurface weaknesses which may lead to landslide, slump or settlement failures. Particularly, an investigation shall be conducted where the stability will be lessened by the grading or where any of the following conditions are discovered or proposed:

1. At fault zones where past land movement is evidenced by the presence of fault gouge.
2. At contact zones between two or more geologic formations.
3. At zones of trapped water or high water table quite often associated with conditions 1 and 2 above.
4. At bodies of intrusive materials.
5. At historic landslides or where the topography is indicative of prehistoric landslides.
6. At adversely sloped bedding planes, short range folding, overturned folds, etc.
7. At locations where a fill slope is to be placed above a cut slope.
8. At proposed cuts exceeding 25 feet in height unless in competent rock or of lesser heights in rock of questionable stability.
9. At the locations of all proposed fills.
10. Where any side hill fills are proposed.
11. Wherever water from rainfall, irrigation, private sewage disposal systems, or other probable sources from both the grading project and adjoining

properties is likely to reduce the subsurface stability.

12. Where the proposed grading may adversely affect the existing or future stability of adjoining properties. The investigation shall be sufficient to outline the problems and solutions to these problems.

The soils engineer and engineering geologist shall submit written reports of their findings to the permittee and the design engineer or land surveyor. Their reports shall include but not necessarily be limited to the following minimum data based upon detailed surface and subsurface investigation:

a. The engineering geologist's report shall include a detailed geologic map showing bedrock, soil, alluvium, faults, shears, prominent joint systems, lithologic contacts, seeps or springs, soils or bedrock slumps, landslides or failures and other pertinent geologic features existing on the proposed grading site. Geologic cross-sections, prepared to reasonably depict anticipated geologic substructure, shall also be included in sufficient number and detail. The report also shall include detailed logs of all borings, test pits or other subsurface data obtained during the course of his investigation. The subsurface exploration shall extend to sufficient depth into the bedrock to expose the deepest rock affecting the proposed grading. The report shall include specific details and observations for the soils engineer's use in analysis of the stability of cut slopes in zones of shallow or perched subsurface waters that may affect slope stability.

b. The soils engineer's report shall include a map of the proposed grading site showing the locations of all subsurface exploratory test pits or borings. Detailed logs of the test pits or boring, including the approximate locations of all soil or rock samples taken for laboratory testing, shall also be included. In addition, laboratory test results, soil classification, shear strength characteristics of the soils and other pertinent soil engineering data shall be presented.

Sufficient cross-sections and cut and fill slope stability analyses shall be included to substantiate recommendations concerning the vertical height and angle of all slopes on the project.

Other aids in exploratory work may be used but subsurface exploratory work sufficient to support the findings shall be performed.

Both the engineering geologist's and soil engineer's reports shall describe the grading project as to its location, topographic relief, drainage, geologic and soils types present, the grading proposed, the effects of such grading on the site and adjoining properties, and shall contain specific conclusions concerning the feasibility and anticipated future stability of the overall project and an analysis of the property on a lot-by-lot basis. Specific recommendations for the correction of all known and/or anticipated geologic hazards on the grading project must be included.

Subject: Board Ruling--Stilt Supported Buildings Erected on Slopes Exceeding Two Horizontal to One Vertical--RR 22851

Recommendation

Approval for the Department to issue permits for stilt supported dwellings on caissons or piers where located over a fill slope exceeding two horizontal to one vertical. The Superintendent of Building shall determine that good engineering practice would permit the conditional use of such a dwelling subject to compliance with the following conditions and such other precautions found to be reasonable and necessary.

1. All footings shall be designed by a licensed engineer and extend through the fill a minimum of 3'-0" into the underlying bedrock but not less than the depth required to resist the lateral load by friction or passive resistance as determined by the foundation engineer.
2. All caissons shall be reinforced for their full length with a minimum of four No. 4 bars tied with 1/4" hoops at 12" o.c.
3. All caissons or pier footings shall be tied laterally in two directions at the ground surface with grade beams or tie beams a minimum of 12" x 12" in cross-section reinforced with a minimum of four No. 4 bars tied with 1/4" hoops at 12" o.c.
4. All roof drainage is collected and conducted to the street in a non-erosive device.
5. No additional fill from the footing excavation is placed on the slope.
6. All loose brush and debris shall be removed from the site prior to starting construction.
7. The fill placed upon this property is susceptible to downhill creep which must be presumed and allowed for in the design. The designing engineer shall provide support against downhill creep which shall not be less than 1000 lbs. per linear foot acting upon each caisson or pier for the full length of its penetration through the fill. If the designing engineer or the Department finds that a greater force is probable, the design shall be modified accordingly.

The above requirements do not preclude consideration of other design methods if performed by an engineer versed in soil mechanics; and if the design is based upon exploratory evidence substantiated by engineers who are approved by the Board to make such investigations.

Exception: Where there is no fill or fill is less than 12" in depth, caissons or piers shall be designed to resist a minimum horizontal force of 1000 lbs. acting downhill on each caisson or other type of footing. Caissons or piers shall be tied together in two directions by grade beams as required in Item No. 3.

8. The site shall be planted as required by the Department to prevent surface erosion.
9. Items 1, 2, 3 and 7 listed above may be omitted if continuous footings are used throughout. Continuous footings shall be reinforced with a minimum of two No. 4 bars at top and bottom of the footing.

APPENDIX B: VENTURA COUNTY LAND DEVELOPMENT MANUAL (reproduced in part)

CHAPTER 7: GRADING

7000. General. All grading for land development is subject to the Ventura County Ordinance Code (UBC Chapter 70). Although grading plans are required as part of the improvement plan package, the plan check fees, agreements, bonding, inspection and certifications are handled under the provisions of the Grading Ordinance. Appurtenances to grading (i.e., drainage devices, fences, walls, etc.) must conform to the Standard Land Development Specifications.

7107. Preliminary Grading Plan. The Developer may desire to accomplish some grading of the site prior to approval of the grading plans. In this case the grading plan may be approved, and a grading permit issued on a preliminary basis. Soils and geologic reports will be required and all other conditions of approval of a grading plan must be met. Grading plans processed in this manner must bear the following statement: CAUTION: PRELIMINARY GRADING PLAN. This plan is approved as a preliminary grading plan only. This approval does not include approval for placement of base materials, or construction of curb and gutter or any other street improvement. Grades are subject to change before approval of the road improvement plans. This note must be removed by change order at the time the road improvement plans are submitted for approval.

7108. Modification to Requirements of the Grading Ordinance. Modification of engineering requirements of the Grading Ordinance, such as steeper slopes or use of rock in shallow fills, will be made only on the basis of soils engineering reports, geological reports, etc., including recommendations for grading procedures and design criteria. Such reports must include calculations, where appropriate, allowing a quick check by County personnel. Anticipated modifications should be indicated at the tentative map stage prior to engineering design. Approval of modifications shall be obtained prior to the issuance of a grading permit for either a grading plan or a preliminary grading plan.

7109. Caution in Regard to Cut/Fill Line. Where a cut/fill line crosses a building pad, see UBC Section 29-03(e) as modified by the Ventura County Ordinance Code.

7110. Engineering Geology and Soils Engineering Reports. Engineering geology and soils engineering reports must be submitted if required by the Building Official (UBC Sections 7006(e) and (f)). Reports required by the Building Official must be submitted through the developer's engineer. Three copies of each report required plus one grading plan must be submitted to Subdivision Engineering for review . County review of such reports shall be transmitted to the Engineer as well as the Soils Engineer and Engineering Geologist, as applicable.

The following criteria are for determining whether soils and geologic reports are required:

1. A soils engineering report may be required if:

   a. The depth of cut or fill is 3 feet or greater, or
   b. The fill is to support structural footings, or
   c. An engineered cut or fill is required.

2. An engineering geology report as well as a soils engineering report may be required for projects in hillside areas and in other areas within the County wherein the County Staff Engineering Geologist believes geologic hazards may exist. A hillside area is defined as one where any of the following conditions exist or are proposed within the project area or the area of any off-site work in connection with the proposed project:

   a. Finish cut or fill slope faces with vertical heights in excess of 10 feet.
   b. Existing slope faces steeper than 10 horizontal to 1 vertical, having a vertical height in excess of 10 feet.

7111. Employment of Engineering Professionals. The owner of land on which engineered grading is to be performed shall execute an agreement with the County to provide professional services. Such agreement shall be acknowledged by each of the professionals involved.

7112. Responsibilities of Engineering Professionals. The Engineering Professionals employed by the property owner on grading work will include the Civil Engineer, the Soils Engineer and the Engineering Geologist. The Civil Engineer's duties will include:

1. Preparation of the grading plan.
2. Design of surface drainage, irrigation and other surface features.
3. Survey and staking of the work.
4. Coordination of the other engineering professionals.
5. Provide "Rough Grading and Final Grading Certification."
6. Preparation of the "As-Built" grading plan.
7. Representing the owner for contacts by the County.
8. Certification of "As-Built" grading plan.
9. Perform such other work as is necessary to comply with the ordinance and to insure proper completion of the work in accordance with good engineering practice.

The Soils Engineer's duties will include:

1. Investigation and report on existing soil conditions.
2. Advising the Civil Engineer on soils problems affecting grading.
3. Inspection and testing of soils moved, exposed, disturbed or processed during construction. The Soils Engineer or his representative shall be on the site at all times when grading is in progress.
4. Testing completed soil masses to determine building foundation requirements.
5. Certifying that the plans and specifications are in conformance with his recommendations and to the final acceptability of the grading.

6. Design of subdrainage, erosion control, buttresses, and other soil connnected features.

7. Perform such other work as is necessary to comply with the ordinance and to insure proper completion of the work in accordance with good engineering practice.

The Engineering Geologist's duties include:

1. Investigation, mapping, and report of existing geological conditions.

2. Advising the Civil Engineer and Soils Engineer on geological conditions which may affect grading.

3. Reviewing geological conditions during construction to see if modification of the grading plan is required.

4. Certifying that the plans and specifications are in conformance with his recommendations and the final grading is stable in regard to geological conditions.

5. Perform such other work as is necessary to comply with the ordinance and to insure proper completion of the work in accordance with good engineering geological practice.

As each of the engineering professionals employed in grading has a responsibility for certification of the work on completion of the project, none of the engineering professionals should be changed during the course of the project. If a change occurs, the new engineering professional must satisfy himself as to the work performed by his predecessor through certifications from his predecessor, field review, soil explorations and testing, or combinations of these or by other methods so that he will be able to certify to the entire project on completion. When changes are being made, grading will be stopped until the new professional has agreed to take responsibility for the work.

The Civil Engineer shall sign and place his registration stamp or number on the grading plan. The Soils Engineer and Engineering Geologist shall indicate, by a suitable statement, signature, registration or certification stamp of number and date on a print of the grading plan submitted to the County, that the plan incorporates all recommendations made by them.

7400. Standard Variances from the Code. Sections 7009 through 7012 of UBC allow the Building Official to approve variances from the UBC where such variance is recommended by the Soils Engineer or Engineering Geologist.

CHAPTER 8: CONSTRUCTION

8000. General. When the improvement and grading plans have been signed and the permits issued by the County Surveyor, the County responsibility for control of the land development is transferred from Subdivision Engineering to Construction Inspection. A construction engineer and an inspector will be assigned by the County to watch the construction to insure that the grading and the construction of road improvements meet the minimum requirements of County ordinances and standards. This assignment in no way relieves the developer from the responsibility for inspection and supervision of construction, or of any responsibility for meeting the requirements of the plans, permits, Grading Ordinances, and the Standard Land Development Specifications or for assuming construction in accordance with recommendations of the Soils Engineers and Engineering Geologist.

8300. Grading Inspection. Inspection of grading is accomplished under the Grading Ordinance. It is emphasized that the Grading Ordinance is directed particularly to grading of private property, and that the responsibilities of the Developer, Developer's Engineer, Developer's Soils Engineer, and Developer's Engineering Geologist are assigned under the Grading Ordinance. Omissions from the plans of any work required by the Grading Ordinance will not excuse the developer from any responsibility for compliance.

8306. Grading Reports. The Building Official requires that the compaction test data, including results, location and elevation, be available for inspection on the site at all times during business hours; or, reports are to be mailed daily to the Building Official's designated representative. The method of reporting shall be determined at the preconstruction conference at the option of the Soils Engineer.

The Building Official requires sufficient inspection by the Engineering Geologist to assure that all geologic conditions have been adequately considered. Where geologic conditions warrant, the Building Official may require interim geologic reports. These reports may be required to include, but need not be limited to reporting, inspection of cut slopes, canyons during clearing operations for groundwater and earth material conditions, benches prior to placement of fill, and possible spring locations.

8309. As-Built Grading Plans. Upon completion of the grading work, the Civil Engineer shall prepare an "As-Built" grading plan. The Soils Engineer and the Engineering Geologist shall indicate by a suitable statement, signature, and date on a print of the "As-Built" grading plan that it agrees with the results of the work for which they were responsible as determined by field inspection. The Civil Engineer shall indicate on the reproducible copy of the "As-Built" grading plan that he has received from the Soils Engineer and the Engineering Geologist and has submitted to the County the signed prints of the grading plans prepared by them. The Civil Engineer shall also sign the reproducible "As-Built" grading plan, certifying that it is correct.

GRADING CONTRACTOR CERTIFICATION

Job Address or
Tract No. ______________________ Locality ____________________

Owner _________________________ Permit No. __________________

I certify that the grading was done in accordance with the plans and specifications, the grading ordinance, and the recommendations of the Civil Engineer, Soils Engineer and Engineering Geologist. It is understood that this certification includes only those aspects of the work that can be determined by me, as a competent grading contractor, without special equipment or professional skills.

Grading Contractor ______________________________________

License No. ____

Instructions: The owner shall sign if the grading was not done by a licensed grading contractor.

APPENDIX C: RECOMMENDED GUIDELINES FOR PREPARING ENGINEERING GEOLOGIC REPORTS

The following guidelines are required for engineering geologic reports submitted to the Department of Public Works, County of Ventura. This information was originally printed in California Geology, November 1974. These guidelines are an example of "state of the art," and all the elements should be considered during the preparation and review of geologic reports. Item V was provided by the Southern California Section, Association of Engineering Geologists; the State Building Safety Board; and the California Division of Mines and Geology.

I. GEOLOGIC MAPPING

A. Each report must be a product of independent geologic mapping of the subject area at an appropriate scale and in sufficient detail to yield a maximum return of pertinent data. In connection with this objective, it may be necessary for the geologist to extend his mapping into adjacent areas.

B. All mapping should be done on a base with satisfactory horizontal and vertical control--in general a detailed topographic map. The nature and source of the base map should be specifically indicated. For sub-divisions, the base map should be the same as that to be used for the tentative map or grading plan.

C. Mapping by the geologist should reflect careful attention to the lithology, structural elements, and three-dimensional distribution of the earth materials exposed or inferred within the area. In most hillside areas these materials will include both bedrock and surficial deposits. A clear distinction should be made between observed and inferred features and relationships.

D. A detailed large-scale map normally will be required for a report on a tract, as well as for a report on a smaller area in which the geologic relationships are not simple.

E. Where three-dimensional relationships are significant but cannot be described satisfactorily in words alone, the report should be accompanied by one or more appropriately positioned structure sections.

F. The locations of test holes and other specific sources of subsurface information should be indicated in the text of the report or, better, on the map and any sections that are submitted with the report.

II. GENERAL INFORMATION

Each report should include definite statements concerning the following matters:

A. Location and size of subject area, and its general setting with respect to major geographic and geologic features.

B. Who did the geologic mapping upon which the report is based, and when the mapping was done.

C. Any other kinds of investigations made by the geologist and, where pertinent, reasons for doing such work.

D. Topography and drainage in the subject area.

E. Abundance, distribution, and general nature of exposures of earth materials within the area.

F. Nature and source of available subsurface information. Suitable explanations should provide any technical reviewer with the means for assessing the probable reliability of such data. (Subsurface relationships can be variously determined or inferred, for example, by projection of surface features from adjacent areas, by the use of test-hole logs, and by interpretation of geophysical data, and it is evident that different sources of such information can differ markedly from one another in degree of detail and reliability according to the method used.)

III. GEOLOGIC DESCRIPTIONS

The report should contain brief but complete descriptions of all natural materials and structural features recognized or inferred within the subject area. Where interpretations are added to the recording of direct observations, the bases for such interpretations should be clearly stated.

The following check list may be useful as a general, though not necessarily complete, guide for descriptions:

A. Bedrock--igneous, sedimentary, metamorphic types.

1. Identification as to rock type (e.g., granite, silty sandstone, mica schist).
2. Relative age, and, where possible, correlation with named formations (e.g., Rincon formation, Vaqueros sandstone).
3. Distribution.
4. Dimension features (e.g., thickness, outcrop breadth, vertical extent).
5. Physical characteristics (e.g., color, grain size, nature of stratification, foliation, or schistosity, hardness, coherence).
6. Special physical or chemical features (e.g., calcareous or siliceous cement, concretions, mineral deposits, alteration other than weathering).
7. Distribution and extent of weather zones; significant differences between fresh and weathered rock.
8. Response to natural surface and near-surface processes (e.g., raveling, gullying, mass movement).

B. Structural features--stratification, foliation, schistosity, folds, zones of contortion or crushing, joints, shear zones, faults, etc.

1. Occurrence and distribution.
2. Dimensional characteristics.
3. Orientation, and shifts in orientation.
4. Relative ages (where pertinent).
5. Special effects upon the bedrock. (Describe the conditions of planar surfaces).
6. Specific features of faults (e.g., zones of gouge and breccia, nature of offsets, timing of movements); are faults active in either the geological sense or the historical sense?

C. Surficial (unconsolidated) deposits--artificial (manmade) fill, topsoil, stream-laid alluvium, beach sands and gravels, residual debris, lake and pond sediments, swamp accumulations, dune sands, marine and nonmarine terrace deposits, talus accumulations, creep and slopewash materials, various kinds of slump and slide debris, etc.

1. Distribution, occurrence, and relative age: relationships with present topography.
2. Identification of materials as to general type.
3. Dimensional characteristics (e.g., thickness, variations in thickness, shape).
4. Surface expression and correlation with features such as terraces, dunes, undrained depressions, anomalous protuberances.
5. Physical or chemical features (e.g., moisture content, mineral deposits, content of expansible clay minerals, alteration, cracks and fissures, fractures).
6. Physical characteristics (e.g., color, grain size, hardness, compactness, coherence, cementation).
7. Distribution and extent of weathered zones, significant differences between fresh and weathered material.
8. Response to natural surface and near-surface processes (e.g., raveling, gullying, subsidence, creep, slope-washing, slumping and sliding).

D. Drainage--surface water and groundwater.

1. Distribution and occurrence (e.g., streams, ponds, swamps, springs, seeps, subsurface basins).
2. Relations to topography.
3. Relations to geologic features (e.g., previous strata, fractures, faults).
4. Sources and permanence.
5. Variations in amounts of water (e.g., intermittent springs and seeps, floods).
6. Evidence for earlier occurrence of water at localities now dry (e.g., vegetation, mineral deposits, historic records).
7. The effect of water on the properties of the in-place materials.

E. Features of special significance (if not already included in foregoing descriptions).

1. Features representing accelerated erosion (e.g., cliff reentrants, badlands, advancing gully heads).

2. Features indicating subsidence of settlement (e.g., fissures, scarplets, offset reference features, historic records and measurements).

3. Features indicating creep (e.g., fissures, scarplets, distinctive patterns of cracks and/or vegetation, topographic bulges, displaced or tilted reference features, historic records and measurements).

4. Slump and slide masses in bedrock and/or surficial deposits, distribution, geometric characteristics, correlation with topographic and geologic features, age and rates of movement.

5. Deposits related to recent floods (e.g., talus aprons, debris ridges, canyon-bottom trash).

6. Active faults and their recent effects upon topography and drainage.

IV. THE BEARING OF GEOLOGIC FACTORS UPON THE INTENDED LAND USE

Treatment of this general topic, whether presented as a separate section or integrated in some manner with the geologic descriptions, normally constitutes the principal contribution of the report. It involves both (1) the effects of geologic features upon the proposed grading, construction, and land use, and (2) the effects of these proposed modifications upon future geological processes in the area.

The following check list includes the topics that ordinarily should be considered in submitting discussion, conclusions, and recommendations in the geologic reports:

A. General compatibility of natural features with proposed land use: Is it basically reasonable to develop the subject area?

1. Topography.
2. Lateral stability of earth material.
3. Problems of flood inundation, erosion, and deposition.
4. Problems caused by features or conditions in adjacent properties.
5. Other general problems.

B. Proposed cuts.

1. Prediction of what materials and structural features will be encountered.

2. Prediction of stability based on geologic factors.

3. Problems of excavation (e.g., unusually hard or massive rock, excessive flow of groundwater).

4. Recommendations for reorientation or repositioning of cuts, reduction of cut slopes, development of compound cut slopes, special stripping above daylight lines, buttressing, protection against erosion, handling of seepage water, setbacks for structures above cuts, etc.

C. Proposed masses of fill.

1. General evaluation of planning with respect to canyon-filling and sidehill masses of fill.

2. Comment on suitability of existing natural materials for fill.

3. Recommendations for positioning of fill masses, provision for underdrainage, buttressing, special protection against erosion.

D. Recommendations for subsurface testing and exploration.

1. Cuts and test holes needed for additional geologic information.

2. Program of subsurface exploration and testing, based upon geologic considerations, that is most likely to provide data needed by the soils engineer.

E. Special recommendations:

1. Areas to be left as natural ground.
2. Removal or buttressing of existing slide masses.
3. Flood protection.
4. Protection from wave erosion along shorelines.
5. Problems of groundwater circulation.
6. Position of structures with respect to active faults.

V. SEISMIC CONSIDERATIONS

The following published guidelines should be considered when preparing seismic information.

1. California Department of Mines and Geology Note No. 37, "Guidelines to Geologic/Seismic Reports."

2. California Department of Mines and Geology Note No. 43, "Recommended Guidelines for Determining the Maximum Credible and the Maximum Probable Earthquakes."

VI. DOCUMENTATION AND IMPLEMENTATION

A. The report should consider as the minimum requirement Chapter 70, Uniform Building Code (1973). Refer to California Administration Code, Title 25, Section 1090, Excavation and Grading.

B. All material in the report should be relevant to the purpose of the report.

C. All statements should be documented by references or by accurate field observations.

D. Areal photos (originals or suitable copies) should be included to document any discussion on landslides and faults.

E. The method(s) of field analysis should be discussed in a lucid manner.

LANDSLIDING AND FLOODING IN SOUTHERN CALIFORNIA DURING THE WINTER OF 1979-80

by F. Harold Weber, Jr.

The six storms that swept through southern California during the period February 13-21, 1980, caused widespread damage in southern California. Some of the areas damaged, such as the Santa Monica Mountains and Puente Hills, suffer damage during every winter with prolonged intense rainfall, but probably suffered less damage in 1980 than in 1978. Others, however, such as the Monterey Park area in the Repetto Hills, Bradbury in the foothills of the San Gabriel Mountains, and communities of coastal San Diego County, suffered more damage from shallow debris slides and flows in 1980 than in 1978. Damage may have been less in the Santa Monica Mountains, Puente Hills, and Baldwin Hills in 1980 than in 1978 partly because the 1978 rains removed most of the soil and colluvium (slopewash) from the sites most vulnerable to sliding. Additionally, the areas more seriously damaged in 1980 than in previous years may not have received a sufficient amount of sustained intense rainfall to have triggered widespread landsliding since their development 30 to 40 years ago.

The improvement in grading codes and tract development techniques in the last 15 or more years has considerably lessened the chances for damage to more recently constructed buildings in southern California, but landsliding in 1978 and 1980 also damaged many recently developed properties.

INTRODUCTION

In mid-February 1980 a series of six storms swept through southern California, bringing nearly 13 in. of rain in nine days to downtown Los Angeles and more than double that amount to hilly and mountainous areas. The resulting landslides and floods caused an estimated $400 million in damage to homes, businesses, agricultural lands, and public facilities.

F. Harold Weber, Jr., is Senior Geologist with the California Division of Mines and Geology, Los Angeles, California.

Note: Adapted from a report of the same title prepared for the California Division of Mines and Geology (Weber, 1980).

Hardest hit by landslides was the City of Monterey Park in the southern Repetto Hills (Figure 1). Landslides and debris flooding also caused heavy damage in parts of the Santa Monica Mountains and Puente Hills in Los Angeles County, in the Ventura-Oxnard area of Ventura County, in southern Orange County, in the coastal region of San Diego County, and in the City of San Bernardino in San Bernardino County, where the Harrison Canyon debris basin overflowed. In Riverside County the San Jacinto River overflowed, flooding the City of San Jacinto and causing Elsinore Lake to rise above its outlet level and flood developments around the lake and below its outlet.

More damage may have been caused in southern California by the February 1980 rains than by either the exceptionally heavy 1978 or 1969 rains. This is primarily because the heavy rainfall during the 1980 rains was concentrated over a wider region of southern California than it was during the two previous years of exceptional rain.

GEOLOGIC ASPECTS

The 1980 rains apparently caused more damage in southern California than did the rains of 1978, and also probably more damage than did the rains of 1969. But a greater proportion of the damage in 1980 was caused by water and debris flooding than in 1969 and 1978, when landslides seem to have been a more dominant cause of damage than in 1980. In 1980 landslides were a substantial cause of damage only in selected areas and a cause of calamitous damage only in the Monterey Park area of the Repetto Hills in Los Angeles County (Figure 2).

Areas relatively free of damaging landslides in the recent past that suffered at least moderate damage from landslides in 1980 are Bradbury, about 12 miles northeast of Monterey Park, and parts of the central coastal San Diego region (Oceanside, Carlsbad, Encinitas, Rancho Bernardo, and Poway). Graded slopes of both older and newer tracts of the Puente Hills that suffered widespread damage in 1978 also were damaged by the 1980 rains, but not to the extent of 1978. Only a few localities were seriously damaged in 1980 in the Baldwin Hills, where widespread damage occurred in 1978. Landslides were locally common in the Santa Monica Mountains in 1980 but apparently caused less damage than in 1969 (Campbell, 1975) and in 1978 (Weber et al., 1978). Shallow debris slides and bedrock landslides occurred along the Malibu-Pacific Palisades coast (Figure 3).

The total dollar loss from damage caused by water and debris flooding was large in 1980, but probably only because very large parts of southern California received voluminous rainfall and runoff during the principal nine-day storm period and at other times. In 1969 and 1978 smaller parts of southern California suffered the most intense landsliding and flooding (such as the Ventura-Santa Monica Mountains region in 1969 and the southwestern San Gabriel Mountains in 1978). In 1980, however, intense flooding and landsliding occurred in such widely separated regions as Ventura, the Santa Monica Mountains, San Jacinto-Elsinore Lake, Palm Springs-Coachella Valley, San Clemente, and Rancho Bernardo-Carlsbad (the La Costa area).

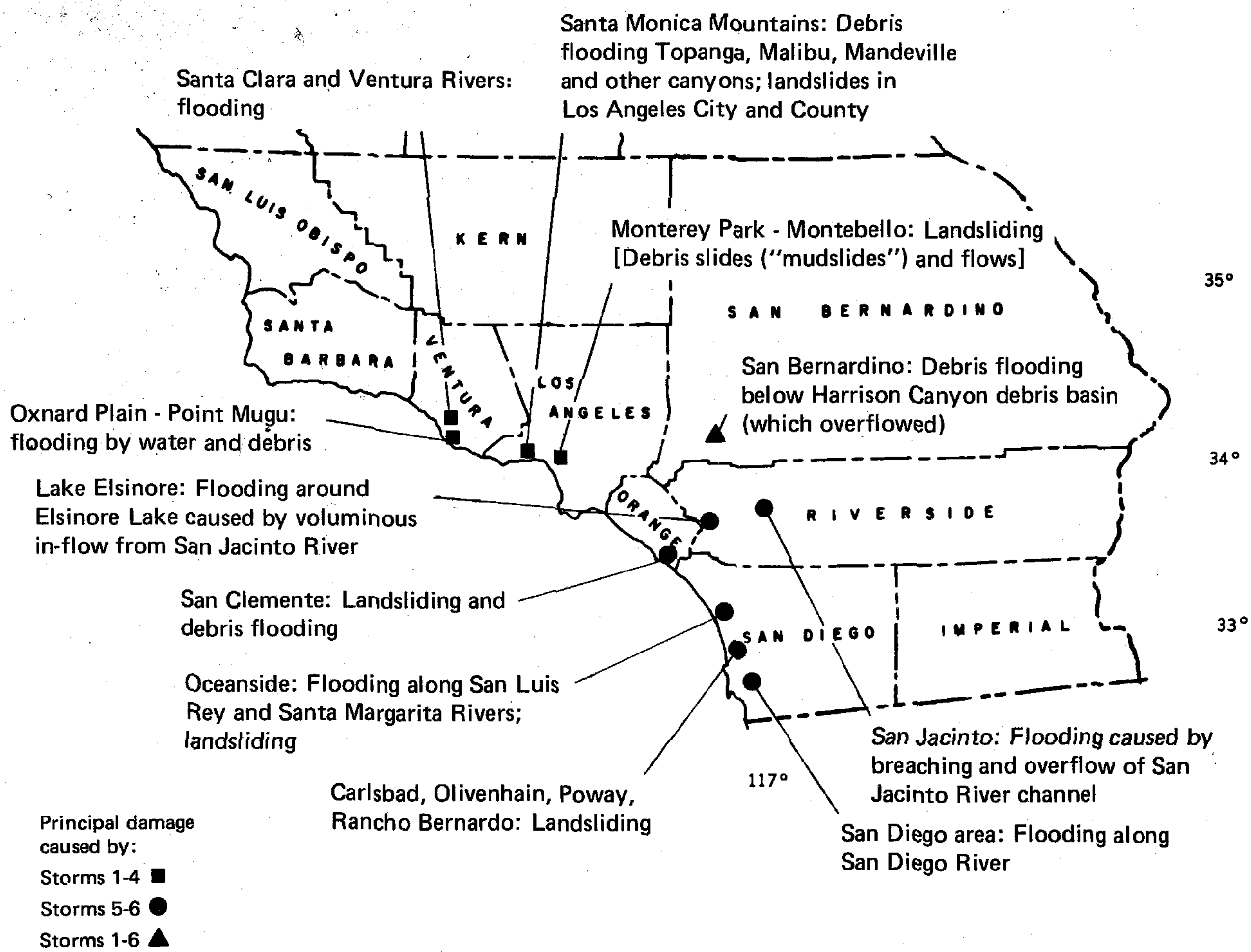


FIGURE 1 Map of southern California showing principal areas or localities that received damage from 1980 rains.

Apparently anomalously, southern California has had intense, destructive rains in two of the last three years, but this appears to be unusual only when considering very recent history. Southern California had periods of closely spaced rainy years during the 1930s and early 1940s: for example, the winters of 1934-35, 1936-37, 1937-38, 1939-40, and 1940-41 yielded 19 in. or more of rain at the Los Angeles Civic Center. *Yet records show that only 29 of the* 103 winters from 1878-79 to 1980-81 (the period for which records have been kept) have yielded at least 19 in. of rain at the Civic Center (and proportionally larger amounts, up to twice as much or more, in higher regions such as the Santa Monica and San Gabriel mountains). Other periods were relatively dry: for example, during the 20-year period including the winters of 1944-45 and 1964-65 only the winters of 1951-52 and 1957-58 yielded 19 in. of rain or more at the Civic Center. (The writer considers the figure of 19 in. to be the apparent threshold for total annual rainfall at the Los Angeles Civic Center to indicate those years when moderate to severe landsliding and

FIGURE 2 Photograph showing path of debris flow that seriously damaged the house at right on Divina Vista Street in Monterey Park.

flooding may occur in southern California.) A long-range view of annual rainfall data for the 103-year period noted above indicates that the complete cycles from dry through wet to dry are about 25 years in length. On that basis, the trend of the data in 1978 indicated that the region was leaving a relatively drier period and entering a wetter period (as previously stated by Weber et al., 1978, p. 22).

The lack of widespread severe landsliding in 1980 in such areas as the Santa Monica Mountains and Santa Paula, as contrasted with 1969, and in the Baldwin Hills area, as contrasted with 1978, might be explained as follows.

Rainfall during the principal nine-day storm period of 1980 may not have been prolonged enough, or spaced out sufficiently, in most areas for the ground first to have become saturated by gentle to moderate rainfall and then to have been struck by another period of prolonged (roughly from 6 to 12 hours) moderate to intense rainfall--the condition that is apparently needed to trigger regionally widespread shallow debris slides and flows, bedrock and fill slumps, and bedrock glides. Only in the Monterey Park area of the Repetto Hills did this classic two-step rainfall pattern occur and cause

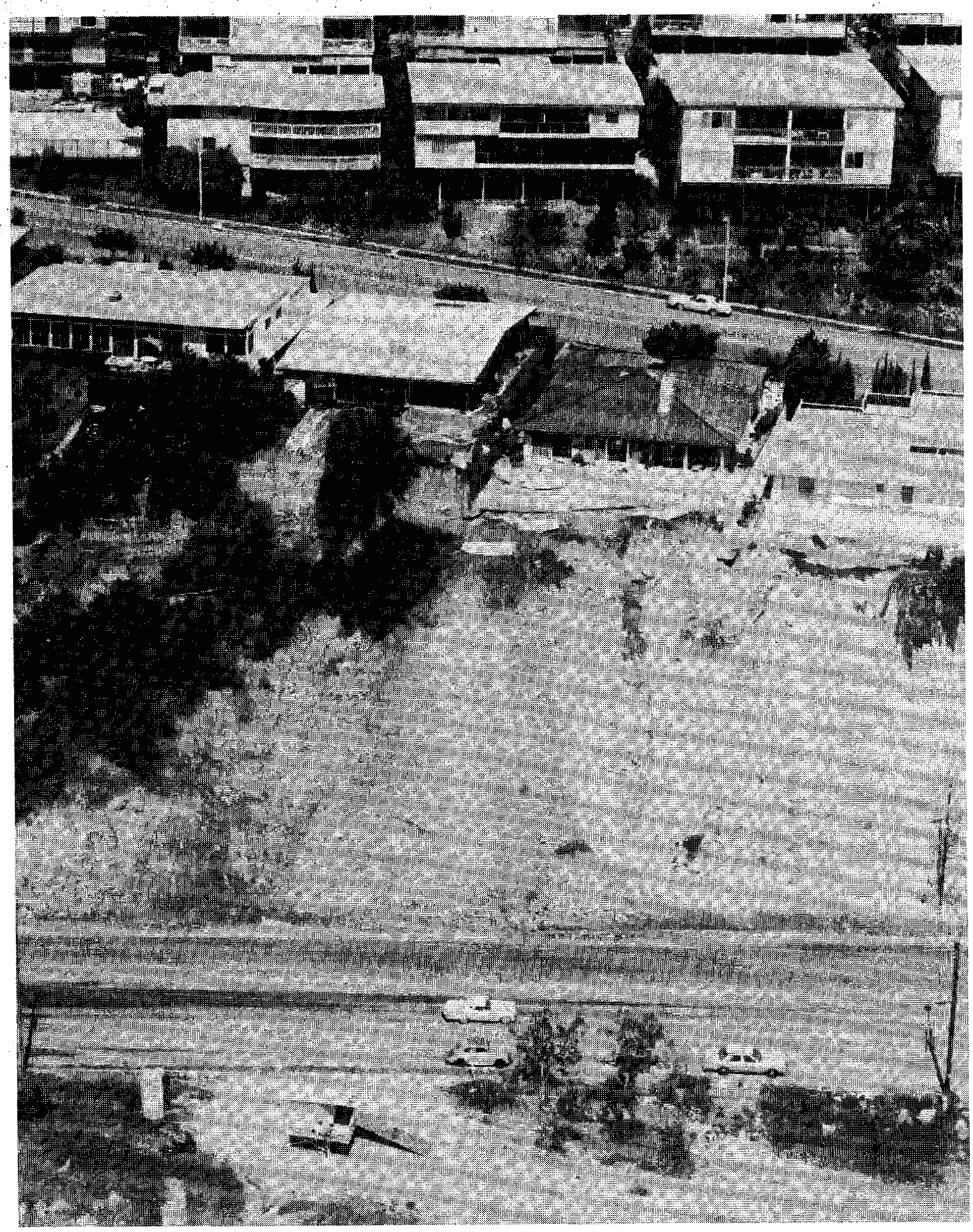


FIGURE 3 Aerial view north across Pacific Coast Highway in Malibu, Los Angeles County, shows sloughing and sliding that has damaged residential properties built too close to the edge of the ancient sea cliff. Rocks along the coast here are commonly highly fractured and deeply weathered and, hence, very susceptible to slope failure. (Photograph courtesy of A. L. Parmer, California Department of Transportation.)

widespread destructive landsliding. There about 6.5 in. of rain fell during two storms that occurred on February 13 and 14, saturating the ground. Then an estimated 5 in. of rain fell in some parts of the city from mid-morning to early evening on February 16, causing debris slides and flows between 5:00 and 8:00 p.m. that damaged more than 100 dwellings (personal communication, Lloyd DeLlamas, City Manager of Monterey Park). These slides and flows stripped away the mantle of soil and colluvium overlying the clayey siltstone-sandstone of the Pliocene Fernando formation that underlies the damaged area. Especially affected was a slope along Divina Vista Street that had not suffered ubiquitous serious damage since development began on it in the late 1930s. Apparently, conditions were right in 1980 for serious widespread slope failure in the Monterey Park area for the first time in more than 40 years. These conditions included: (1) buildup of surficial material on the slope; (2) probable weakening of the slope because of the modification of drainage caused by continuing development over the last 40 years; and (3) intense rainfall falling for a sufficiently lengthy period of time on already saturated ground to cause pore pressure to increase to the point where many slides and flows were triggered.

No doubt, considering the large amount of rain that has fallen on southern California in the last three years, ancient or older historical landslides may become active (and destructive) later in the year (Figure 4). (For example, the destructive Bluebird Canyon landslide in Laguna Beach did not occur until October 1978, more than six months after the preceding intense rains; it is generally believed that the water from the rain percolating into an ancient landslide helped to trigger that 1978 slide [Tan, 1978].)

A second factor may be that a great many of the slopes that developed an unstable condition in the 1970s responded to the sequence of saturating and then intense (triggering) rains in 1978 by sliding, slumping, and flowing so that the unstable material was removed. This appears to have been especially true of the Baldwin Hills, where failures were sparse in 1980 compared with 1978. Conversely, in Monterey Park the rains of 1978 may not have been pervasive and intense enough to loosen steep unstable collections of colluvium, soil, and vegetation, whereas the 1980 rains were. In addition, a relatively substantial amount of time must pass before massive new collections of soil, colluvium, and vegetation can build up to an unstable degree again on slopes that have failed (commonly bare to bedrock) in historical or prehistorical times. (Some slopes, however, failed only partly in 1978 and 1980, thus signaling probable future danger points for more massive and widespread failures.)

SOCIAL-GOVERNMENTAL ASPECTS

Public Utilities

As in most natural disasters, the public utility agencies and companies seem to have been able to restore service to most areas fairly quickly; even in areas severely damaged, most services were restored within two or three days.

FIGURE 4 Damaging bedrock landslide that occurred in mid-April 1980 in the Mount Washington-Glassell Park section of the City of Los Angeles. The landslide probably resulted from the reactivation of an ancient bedrock landslide. (Photograph by John Shadle, Los Angeles City.)

Flood Control

Most of the facilities of the Los Angeles County Flood Control District were reported to have performed well. In addition, the district was able to clean its debris basins as they filled. Only a few basins overflowed, and those were principally in drainages where fires had recently burned upstream. A measure approved by Los Angeles County voters last year to restore revenue lost by the district as the result of Proposition 13 becoming law was generally given credit for providing the district with nearly adequate funds for their cleanup work. Several spokesmen for the district provided excellent status reports on the radio during the rains, assuring or warning the public as to the safety of flood control facilities. Some rustic areas most affected by water and debris flooding, such as the communities of Topanga, Mandeville Canyon, and Monte Nido, have very minimal flood control facilities.

One citizen of the Santa Monica Mountains described erosion and flood control structures in the vicinity of her tract residence that apparently were privately constructed to protect the tract, but for which now there is a question as to who is responsible for their maintenance. The responsibility for maintenance for such structures may need to be fixed and clarified.

In San Bernardino the Harrison Canyon debris basin and accompanying facilities proved woefully inadequate to protect a tract of houses, directly downstream, from water and debris coming from a relatively small canyon that was burned over by a brushfire in 1979.

The Corps of Engineers is planning to construct adequate flood control facilities to protect the Pacific Missile Test Center in Ventura County from Calleguas Creek, which broke through its levee and flooded during heavy runoff. In Riverside County the channel for the San Jacinto River proved inadequate to contain the flow of runoff in 1980; downstream, much permanent or semipermanent development has been allowed around the edge of Elsinore Lake below its apparent safe level (at least above the maximum level attained during this year's rains--nearly 1,266 ft above sea level.)

Assistance to Citizens

In most cases assistance to homeowners and others was generally considered to be expeditious and satisfactory. The Red Cross established its shelters very quickly in many places. "One-stop" centers for people to apply for government relief programs also were established very quickly. Radio and television stations and newspapers continually provided telephone numbers for people to call in their emergency requests to government agencies and utility companies and to request information.

Citizen Response

At least 10 people were killed when they drove into, or attempted to cross by foot, water that was running too swiftly and too deeply in normally dry places. As in 1978 and 1969, people in dwellings and other buildings were injured or killed when slopes behind gave way suddenly and watery masses of mud, silt, sand, rock, and vegetation crashed into or through the buildings.

Even if they are not injured, most people whose houses and belongings are badly damaged or ruined suffer mild to severe psychological problems, and some mental health agencies announced during the aftermath of the rains that they were prepared to give help to people so affected. Psychological problems of adjustment to the sudden calamity appear to be especially serious in communities (such as Monterey Park in 1980 and Baldwin Hills in 1978) that have not suffered previous severe and widespread damage from slope failures or flooding, where residents tend to be unaware of the potential hazards surrounding them. In contrast, residents of Topanga, for example, are relatively used to flooding and landslides. Also, communities downslope or downstream from recently burned areas tend to be relatively aware that it is logical to expect possible problems in heavy rainstorms.

In the Monterey Park area many slopes failed that had been planted with common ice plant, which because it is heavy and has very shallow roots is no longer recommended for planting to protect slopes, and in some areas is banned. In some newer tracts (in Hacienda Heights and in the Puente Hills, for example) slopes have been planted with a lighter ground cover, either by the developer or by the homeowner with instructions from the developer or the local government. Because engineering measures to restore and protect damaged slopes on single lots may cost as much as $50,000 to $100,000 and possibly more, it is vital for homeowners to protect and maintain slopes as well as possible to minimize the potential of failure.

A volunteer group, "The Tree People," originally formed to plant trees, received much praise for its effort to sandbag and help clean up residential property during the 1980 storms, mostly in the Santa Monica Mountains, where they had an officially designated headquarters in a local park facility. The activities of the California Conservation Corps seemed to be less noticeable and were receiving less media attention during the 1980 rains than they had in 1978, but in early March, after the rains, they were reported to be providing help in Monterey Park, Lake Elsinore, and other areas. Los Angeles City Councilman Robert Ronka set up a disaster emergency headquarters at his district office in Sunland-Tujunga where people could seek assistance or volunteer aid.

Some citizens of the Los Angeles City area of the Santa Monica Mountains were outraged when they were issued citations by city inspectors for having dumped debris that had come down from their backyard slopes into city streets in front of their residences. The city had picked up such material without comment after the 1978 rains. By law the city is allowed only to clean up debris from the streets that is placed there naturally by flooding or by debris slides and flows, even though most of this debris is derived from natural and graded slopes that are on private property.* The city later received $500,000 in federal funds to be used to pick up from private property debris that owners could document was derived from the storms.

Many residents of damaged areas interviewed by the press seemed only vaguely aware of the dangers of living in canyons that exist only because through geologic time they have been carved out of bedrock by the strength of voluminous runoff during intermittent heavy rains. Furthermore, many residents are apparently unaware of the implicit dangers of apparently innocent slopes in their neighborhood failing during such rains, including natural slopes that have not failed in historical time and brushy, gently to moderately pitched slopes developed in rocks that appear to be resistant to sliding. Even rocky volcanic terrain can yield debris slides and flows, given the requisite amount and spacing of rainfall, as the writer observed in Agoura in 1969. Many people living in landslide- and flood-prone areas simply do not seem to be aware of what can happen during heavy rains or to have given even brief thought to the possibility of potential geologic hazards.

Building Codes

Grading codes of Los Angeles County and City and those of other counties and cities in southern California have evolved gradually since the early 1950s. Los Angeles County and City adopted sophisticated ordinances in 1962 and 1963, respectively, and have updated them since those years. California State law now requires all counties and cities to use Chapter 70 of the Uniform Building Code as the minimum grading code for hillside development if they do not have their own equal or more stringent code.

Properties along Divina Vista Street in the City of Monterey Park that were seriously damaged by the 1980 rains were developed in the 1930s, before grading codes were developed for any county or city in southern California. Other properties damaged in Monterey Park were developed during the period extending from the 1930s through the 1970s. Although the city has not developed a grading code of its own, it has used Chapter 70 of the Uniform Building Code in recent years; but it still does not require a geologist's input into grading plans. Properties developed in the County and City of Los Angeles during the mid to late 1960s and 1970s (after adoption of modern codes) were also damaged by the 1980 rains: in the county, on Lamplighter Lane and other roads in Malibu, in Calabasas Park, in Liberty Canyon in Agoura, in Diamond Bar, and at other localities; in the city, on Paseo Miramar in Pacific Palisades, on Topeka Drive in Tarzana, in Mandeville Canyon, and at other localities.

New State Legislation

The signing, in mid-March 1980, by Governor Edmund G. Brown, Jr., of Assembly Bill 1571, which was authored by Assemblywoman Gwen Moore of Los Angeles, initiated a slope stability study by the California Division of Mines and Geology in the Baldwin Hills, which were severely damaged by slope failures during the March 4-5, 1978, rains (California Division of Mines and Geology, 1980). Assembly Bill 1571 also adds Section 5105 to the Streets and Highways Code, relating to the Improvement Act of 1911. This act authorizes legislative bodies of cities and counties to create special assessment districts for paying the costs and expenses of various improvements to property in the district benefited by the work done. Section 5105 further authorizes "work to prevent, mitigate, abate or control a geologic hazard, as defined, or to repair damages therefrom, and the performance of such work on private property under specified conditions."

Senate Bill 1195 by Senator Robert Beverly was enacted into law in 1979. This law provides a mechanism for public financing of projects for the mitigation, abatement, or control of a geologic hazard through the establishment of "geologic hazards abatement districts."

---

*Proposition 4, approved by state voters on June 3, 1980, will allow a change in these procedures.

RECOMMENDATIONS

The locally disastrous effects of the 1980 (and 1978) rains, viewed in light of indications that the southern California region has entered a period of closely spaced rainy years similar to the wet cycle that occurred in the 1930s and early 1940s, plainly show that more needs to be done to prevent disasters caused by local landsliding and flooding.

In recognition of this need, the Department of Conservation and the Division of Mines and Geology recommend that the state legislature create a program to identify and zone areas of high landslide hazards. The division also recommends that the state legislature create a program to conduct special slope stability studies of particularly sensitive areas, such as the coastal Santa Monica Mountains, for use by local agencies in their administration of codes. In addition, the division recommends a number of measures--many of which are already in effect in some areas--that could be instituted at the local level. These include mandatory notification to prospective buyers of houses or apartment buildings of known slope stability problems; the establishment of assessment districts for upgrading and maintaining the adequacy of slopes (similar to brush control inspections for fire safety) to ensure that drains and vegetation are maintained adequately; more effective mitigation of the hazard of flooding, debris flooding, debris flows, and bedrock landsliding posed by burned-off areas; more stringent enforcement of grading codes, including on-site inspections before, during, and after grading for residential tracts and for individually sited residences and other buildings; improvement of building codes and fill slope engineering; and formal programs to educate people about the hazards of potential landslides and floods.

REFERENCES

California Division of Mines and Geology (1980) Slope Stability Study of the Baldwin Hills, Los Angeles County, California, Open-File Report 80-14 LA, California Division of Mines and Geology, Sacramento, 162 pp.

Campbell, R. H. (1975) Soil Slips, Debris Flows and Rainstorms in the Santa Monica Mountains and Vicinity, Southern California, U.S. Geological Survey Professional Paper 851, U.S. Geological Survey, Reston, Virginia, 51 pp.

Tan, S. S. (1978) Case study: the Bluebird Canyon landslide, Laguna Beach, Orange County, Landslides in the Los Angeles Region, California--Effects of February-March 1978 rains, eds. Weber et al., Open-File Report 79-4 LA (revised 1979), California Division of Mines and Geology, Sacramento, pp. 226-232.

Weber, F. H., Jr. (1980) Landsliding and Flooding in Southern California During the Winter of 1979-1980, Open-File Report 80-3 LA, California Division of Mines and Geology, Sacramento, 69 pp.

Weber, F. H., Jr., Treiman, J. A., Tan, S. S., and Miller, R. V. (1978) Landslides in the Los Angeles Region, California--Effects of February-March 1978 Rains, Open-File Report 79-4 LA (revised 1979), California Division of Mines and Geology, Sacramento, 265 pp.

THE MALIBU SLIDE

by Raymond A. Forsyth and Marvin L. McCauley

What has become well known as the Malibu slide occurred adjacent to the Pacific Coast Highway in the central part of the Santa Monica Mountains one and one half miles west of Topanga Canyon. Sliding in this area is a common occurrence as the result of bluff recession due to oversteepening by sea erosion.

On April 13, 1979, a rock slide resulted in closure of the Pacific Coast Highway. A field review disclosed a system of cracks and fissures above the bluff face, suggesting the possibility of a mass movement of 60,000 cu yd of slide debris onto and across the highway. The primary problem posed by the slide was that of keeping the Pacific Coast Highway at least partially open while ensuring the safety of traffic on the highway during the course of an investigation to develop a recommendation for permanent correction. To accomplish this a barrier suitable for the catchment of small earth and rock falls was erected, along with a monitoring system to provide advance warning of mass movement. The early warning system consisted of the following four elements:

1. Subaudible rock noise stations (SARN)
2. Steel pins for surface measurements
3. Surface extensometers that would trigger an alarm at the roadway level in the event of movements in excess of 0.15 ft in a 24-hour period
4. Continuous visual observation

The investigation to develop a permanent correction consisted of geologic mapping, seismic investigation, vertical and horizontal borings, and a laboratory testing program involving residual shear strength tests on recovered samples. Analysis of the data indicated that a slope flattening necessary to achieve stability would involve removal of 1-1/2 to 3 million cu yd of material. The plan ultimately selected involved removal of 150,000 cu yd plus slope reinforcement with rock dowels.

Raymond A. Forsyth is Chief of the Soil Mechanics and Pavement Branch and Marvin L. McCauley is Senior Engineering Geologist with Caltrans Transportation Laboratory in Sacramento, Calfornia.

The contract for final correction was approved on September 12, 1979. Excavation and rock doweling began in early October. Excavated material was pushed down the face of the slide into the catchment area on a 12-hour-per-day, 6-day-a-week basis. Removal of the slide was accomplished at an average rate of 3,500 cu yd per day, with dowels being installed at an average rate of 30 per day. The correction, which included installation of a steel mesh on the most active part of the bluff face, was completed in June 1980.

GEOTECHNICAL ORIGIN AND REPAIR OF THE BLUEBIRD CANYON LANDSLIDE, LAGUNA BEACH, CALIFORNIA

by F. Beach Leighton

Total damage from the Bluebird Canyon landslide of October 2, 1978, was \$15 million. Although this slide resulted in the destruction of 25 homes, all utilities in the area, and portions of three roads, no serious injuries were sustained. All geologic studies in the area postdated construction of this 20- to 30-year-old residential section of south Laguna Beach.

The slide occupied 1.5 ha and was a portion of a pre-Holocene rock block slide of 2.5 ha with similar lithologic and structural control, namely, a dip slope in sandstone and siltstone of the Topanga formation (middle Miocene). The 1978 slide broke along preexisting discontinuities: (1) a fault zone along the right flank, (2) a former head scarp at the head, and (3) a fracture zone on the left flank. It slid along an inclined bedding surface with a synclinal warp.

The basal rupture surface of the oldest of two major episodes of pre-Holocene sliding lies an average of 6 m below the rupture surface of the 1978 slide, at 27 m vertical depth. These two principal rupture surfaces represent illite-rich plastic clay seams 0.75 to 4 cm thick.

The abnormally heavy 1977-78 rainfall is believed to have percolated deeply into the old slide mass to the impermeable clay seams, permitting saturation and leaching of them, buildup of pore pressures along them, and buildup of a hydrostatic head in the open-structured materials of the large preexisting graben infilling. Cumulative channel entrenchment that removed toe support was a second triggering factor.

F. Beach Leighton is President of Leighton and Associates in Irvine, California.

Note: My thanks are extended to all the members of our geotechnical landslide team at Leighton and Associates on this project, particularly Iraj Poormand, Larry Cann, and Bruce Clark. Dorothy Bergman provided drafting support and Maria Bottoms library assistance on this paper. Their work and that of Brent Taylor in his role of publications editor in spurring and guiding this paper are gratefully acknowledged.

Remedial construction between October 1978 and June 1979 consisted of placing a horseshoe-shaped shear key buttress at the head of the slide and a gravity buttress at the toe. This earthwork involved about 260,000 cu m and was integrated with a subdrain system and a storm drain system. Soldier piles were placed to keep the backslope of the shear key excavation from retreating. They were effective except for one backslope failure that destroyed one residence and undermined another. The reconstruction of the slide area was successful in saving public property and 22 residences around the periphery and in restoring 25 lots without need of further significant grading. New residences have now been built upon most of these 25 lots.

INTRODUCTION

The Bluebird Canyon disaster of October 2, 1978, came unexpectedly to this 20- to 30-year-old residential section of south Laguna Beach, as none of the typical forewarnings had been associated with this event. No serious injuries were sustained, although the landslide occurred while most residents were still sleeping or just rising, between 5:00 and 6:00 a.m. This fortunate record can be attributed to the slow sliding over a period of about 40 minutes, the dominantly translational movement, the restriction of severe internal deformation to the head and foot of the slide, the existence of mostly single-story, wood-frame residences, and the development of scarps and fissures outside most building areas. The horseshoe-shaped main scarp 7 to 10 m in height left six homes overhanging, but the residents were able to escape through unsuspended portions (see Figure 1A).

The stricken area became uninhabitable and was declared a disaster area by federal and state proclamations. Total damage was estimated at $15 million. Twenty-five homes had to be demolished, and access was cut off to other homes by the destruction of utilities (sewer, water, gas, and overhead electricity) and critical portions of three roads (Meadowlark Lane, Meadowlark Drive, and Oriole Drive). Of ominous portent was the blockage of the natural Bluebird drainage channel by the foot of the landslide (Figure 2). This dam, with a reservoir capacity of 12,000 to 19,000 cu m, could have catastrophically failed in the event of subsequent heavy winter rains, resulting in damage to properties downstream and possible isolation of over 280 properties upstream.

Most homes in the slide area were built in the 1950s directly on the natural soils of an undetected preexisting landslide mass. This building program occurred prior to the requirement that engineering geology be applied to residential construction within the incorporated limits of the City of Laguna Beach.

GENERAL GEOLOGIC SETTING

The landslide occurred on the southwest flank of the San Joaquin Hills underlain in this area by the Topanga formation of middle Miocene age. The triangular-shaped cuesta on which the recent and at least two pre-Holocene episodes of sliding occurred is bounded on the south by Bluebird Canyon and on the north and west by Rim Rock Canyon. The modified dip slope of the cuesta has a slope of 6 to 8 degrees with a change to 30 to 36 degrees at the foot

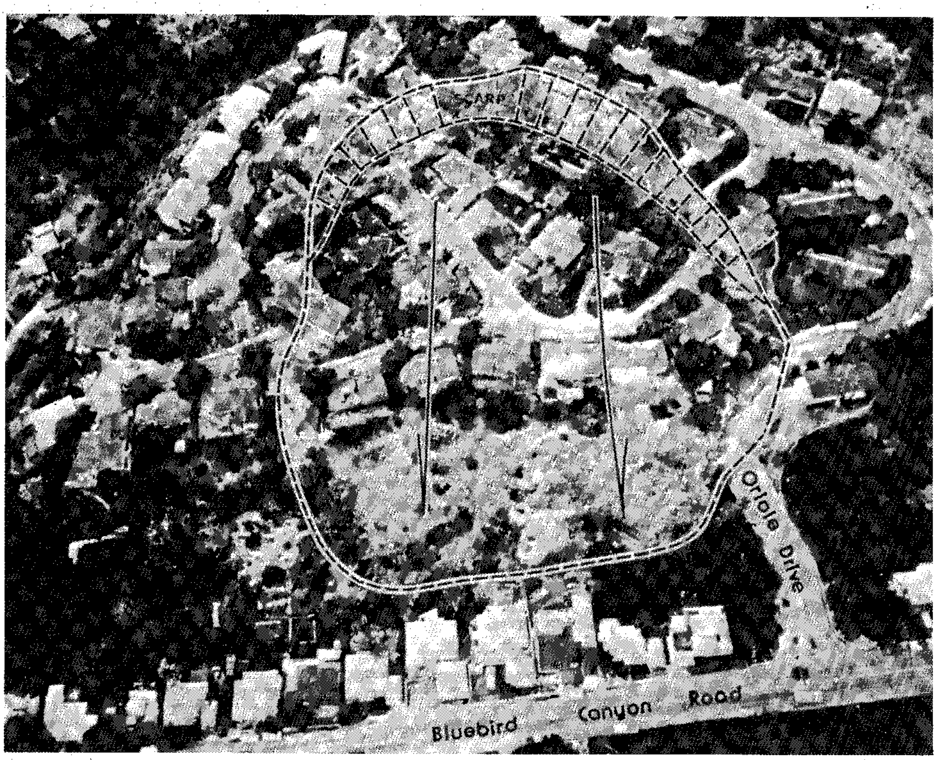


FIGURE 1A Aerial view of Bluebird Canyon landslide immediately following the October 2, 1978, event.

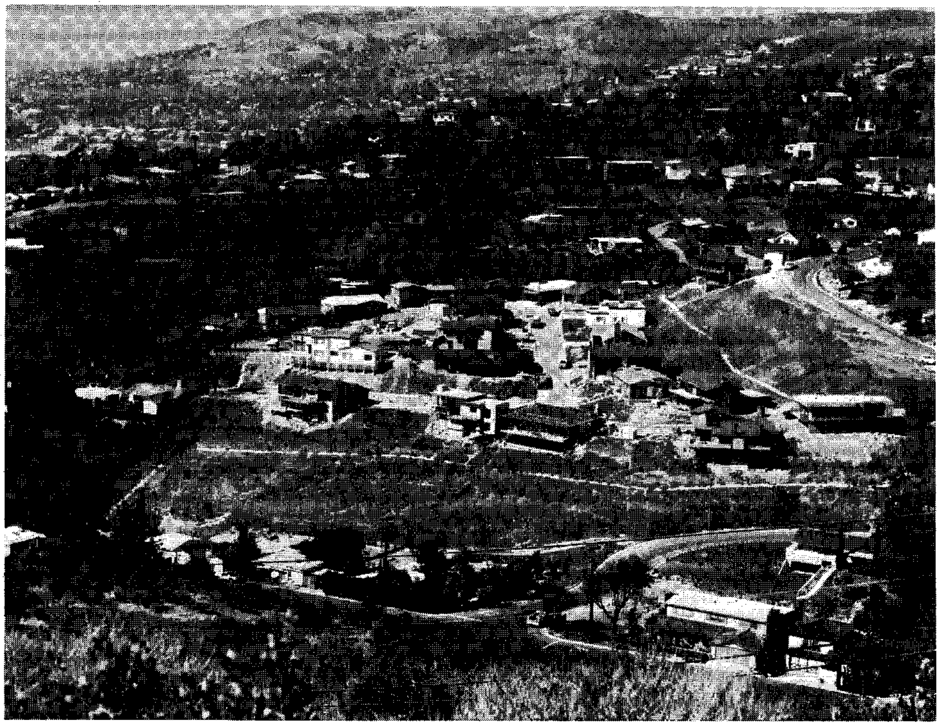


FIGURE 1B View of the same area in 1981 in restored state.

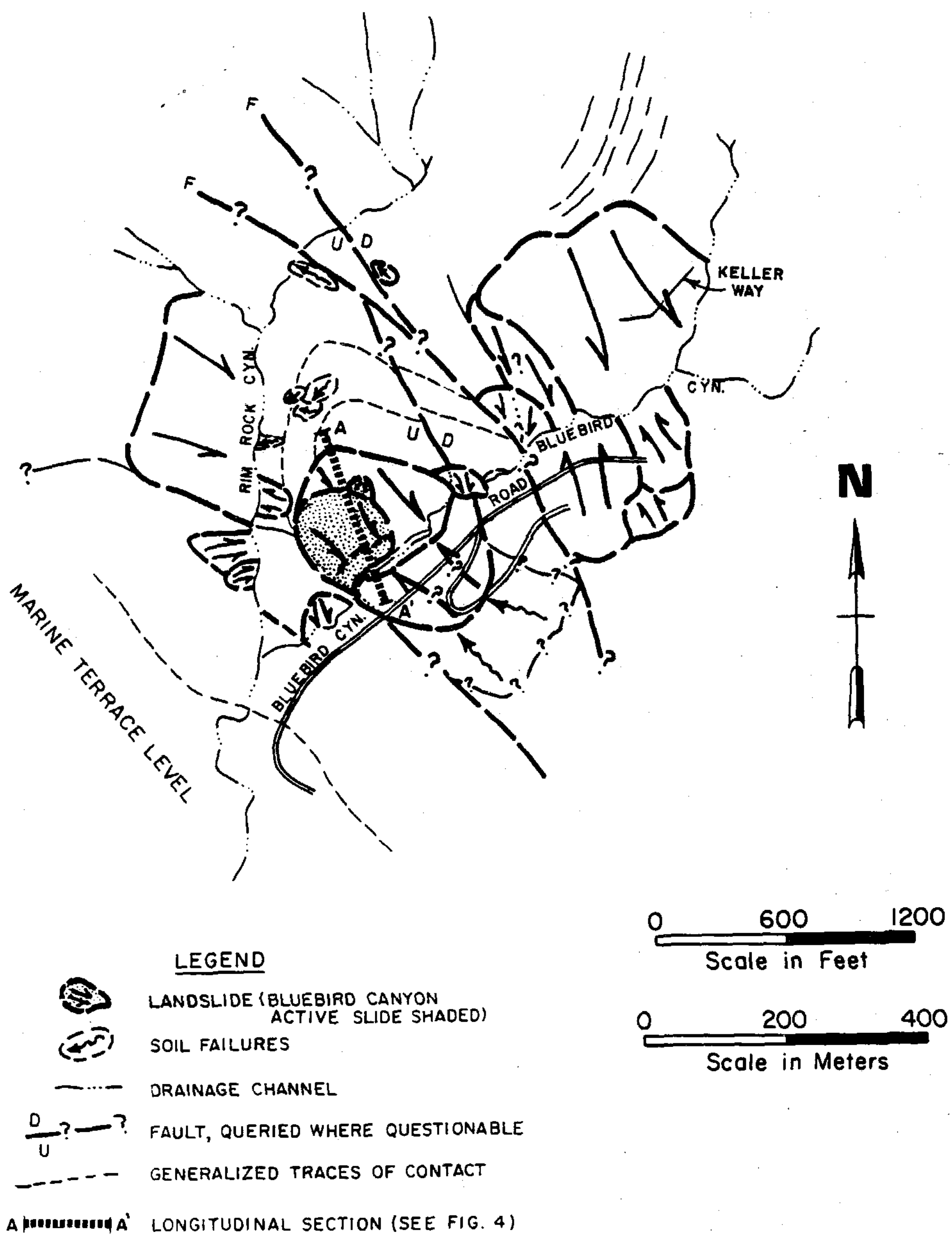


FIGURE 2 Photogeologic index map of Bluebird Canyon area, Laguna Beach.

owing to entrenchment of Bluebird Canyon. The overall median slope angle of 17 degrees approximates the average dip angle of 21°S of the bedrock bedding. As shown in Figure 3, the dip slope is complicated by a synclinal warp in the sandstone-siltstone section and a normal fault zone along the western slide boundary. Subsurface exploration prior to remedial grading and detailed geologic mapping during remedial grading permitted identification of a myriad of small but significant structural features, such as clay gouge zones, joint sets, minor faults and shears, gypsum-filled fractures, and flexural slip surfaces along bedding planes.

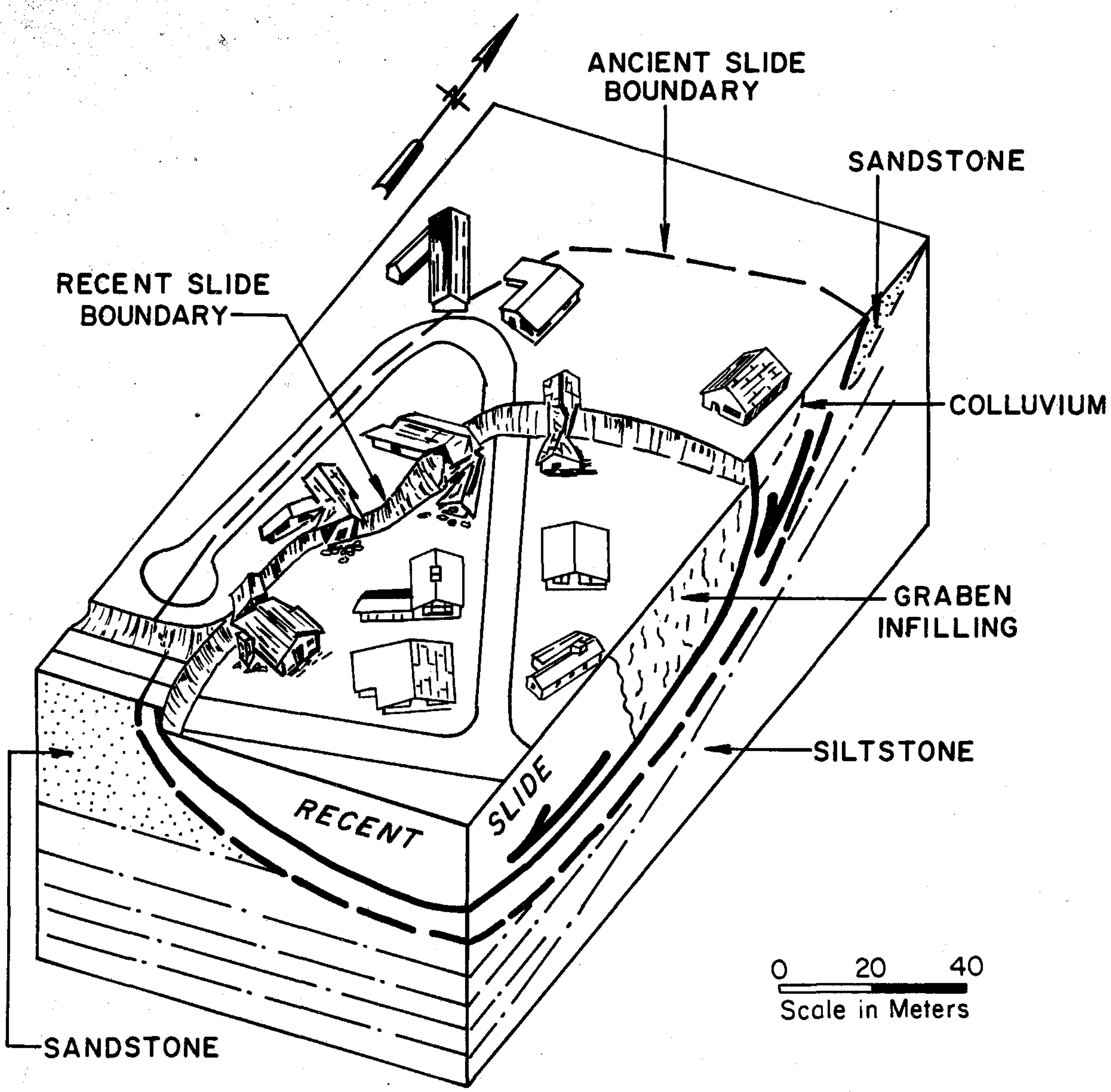


FIGURE 3 Block diagram showing Bluebird Canyon landslide.

GEOTECHNICAL STUDIES

Previous geotechnical studies included a general planning study (Leighton, 1969b) supplemented by a seismic safety element (Leighton and Associates, 1975) and an engineering geologic report of storm damage (Leighton, 1969a). These studies were mostly field and office compilations without detailed original mapping beyond the earlier agency work (Vedder et al., 1957). Geoenvironmental maps of Orange County (Morton et al., 1973) and engineering geologic maps of the Laguna Beach quadrangle (Tan and Edgington, 1976) were important geotechnical contributions, but all of these studies postdated construction of the residences.

Previous published studies did not identify the area as an old slide, other than designating the general area in the 1969 general planning study as a "major bedrock area subject to potential instability."

Studies of the landslide following its October 1978 occurrence were undertaken by Leighton and Associates for the City of Laguna Beach (Leighton and Associates, 1978a,b, 1979). These studies were only partially summarized in published form (Leighton, 1979, 1980). Other published papers include those by Miller and Tan (1979), Sydnor (1979), and Tan (1979).

The principal geotechnical steps taken in the postslide work included:

1. Study of stereoscopic pairs of aerial photographs taken in every decade since the 1920s
2. Field investigation, including geologic mapping on postslide and preslide base maps and aerial photographs
3. Monitoring of postslide deformation at 20 tiltmeter locations, 9 extensometer sites, dozens of cracks and boring offsets, and three slope indicators in the head scarp area for the soldier pile system
4. Measurement of the groundwater regime and geochemical parameters of both natural water and domestic water
5. Clay mineralogy by Gerald Henderson
6. Subsurface logging of 24 holes 0.7 m in diameter ranging in depth from 9 to 35 m, and 45 lineal meters of backhoe excavation
7. Soil sampling and laboratory testing of earth materials
8. Preparation of detailed maps and illustrations, stability analyses, and reports in three stages
9. Remedial geotechnical design and continuous in-grading inspections
10. Compilation of final reports

LANDSLIDE RELATIONSHIPS

The 1978 landslide and the preexisting landslides at the site were rock block slides (block glides) with essentially no rotation of the main block and only slight rotation at the foot (see Figures 3 and 4). The "ancient" slide has an area of 2.5 ha, compared with an area of 1.5 ha for the 1978 slide; the basal rupture surface of the "ancient" slide lies an average of 6 m below that of the 1978 slide, at 27 m vertical depth.

The base of the rupture zone of 1978 was a plastic clay seam that ranged in thickness between 0.75 and 4 cm. This seam was found to be continuous but slightly undulating. X-ray diffraction studies of the clay revealed 50 to 70 percent illite and 25 percent montmorillonite. Conservative soil engineering back calculations indicated a residual angle of internal friction of 9 degrees, with a cohesion of 0.5 tons/sq m.

The rupture surface of the 1978 slide coincides with the base of a large colluvial infilling, interpreted as a pre-Holocene graben (Figure 4). Both the 1978 and the deeper "ancient" rupture surfaces conform to bedding surfaces and are crudely saucer-shaped. However, the deeper rupture surface is more gypsiferous, iron-manganese rich, and coarsely grooved. Because both rupture

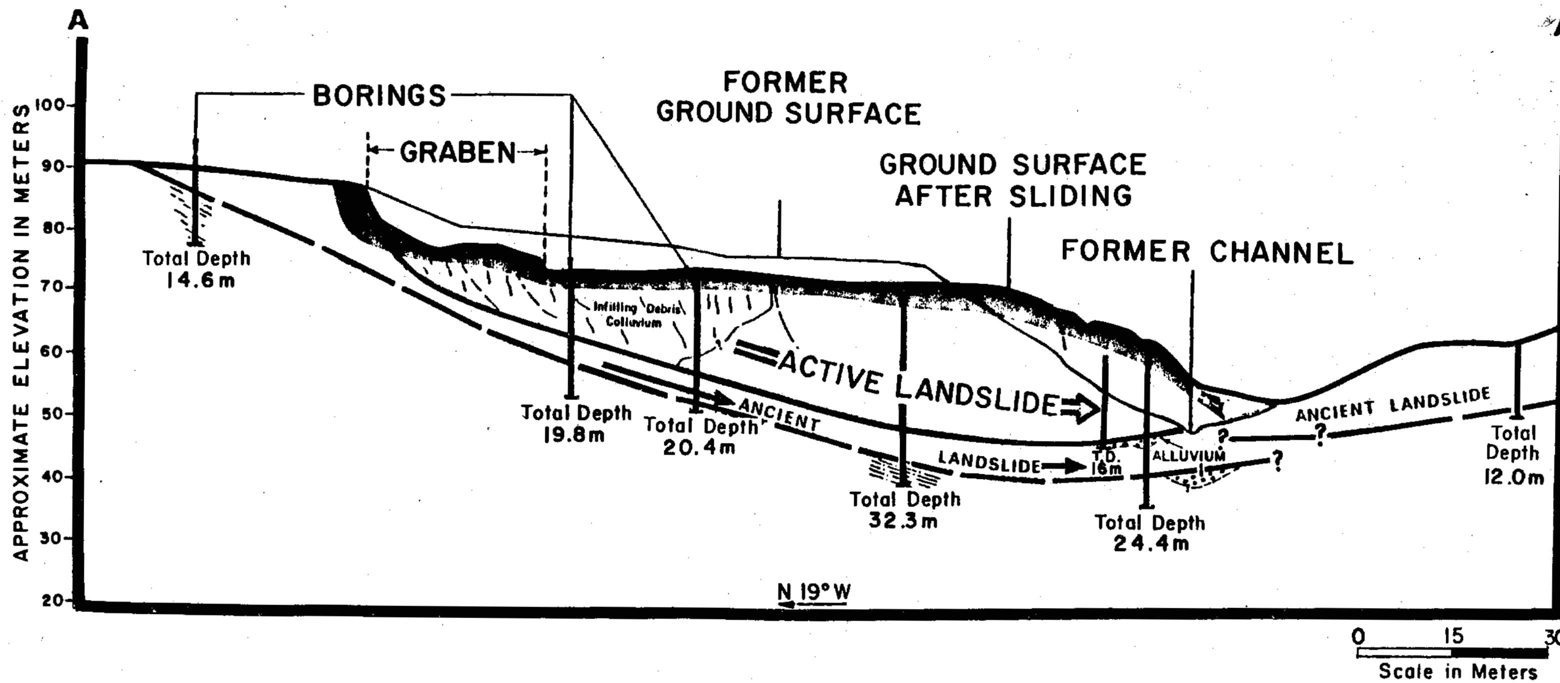


FIGURE 4 Longitudinal section of Bluebird Canyon landslide.

surfaces were buried by a gravity buttress fill and were not exposed in the foot of the slide except in widely spaced borings, radiocarbon dating of earlier landslide episodes was impossible. Stratigraphic and geometric relationships documented at least two pre-Holocene slide events, at least one along the 1978 basal rupture surface and at least one along the "ancient" rupture surface. The "ancient" landslide is believed to be as old as early Pleistocene because its entire head and graben area have disappeared by ridge decapitation.

LANDSLIDE ORIGIN

Four principal factors are believed to have interacted to produce the landslide of October 2, 1978.

1. Clayey siltstone and plastic clay seams of low strength within the Topanga formation dip toward Bluebird Canyon. A preexisting plastic clay seam behaved somewhat like peanut butter in a tilted and soaked sandwich, being striated and grooved as it was overridden.

2. The rock section containing the plastic clay seams was unsupported in the channel area. The slide broke along preexisting discontinuities: a fault zone along the right flank, a former head scarp at the head, and a fracture zone on the left flank. It slid along an adversely inclined bedding surface with a synclinal warp.

3. The Bluebird channel was deepened and surficial mass movements occurred at the foot of the older landslides during the past 50 years. This cumulative channel entrenchment and slope retreat extended from the Pleistocene and culminated in the mass movements and erosion of 1978. A comparison of detailed topographic maps prepared in 1960 and 1978 indicates 5 to 10 ft of downcutting immediately downstream of the 1978 landslide.

4. The abnormally heavy and extended rainfall of 1977-78 percolated into the subsurface, particularly in the large older graben area, to a greater degree than normal. This is evidenced both by a relatively high and flat piezometric surface in the graben area, with a steep gradient beneath the slide mass, and by a substantial dilution of the salinity of groundwater in the graben area relative to the naturally saline groundwater in other parts of the slope. Introduction of the water was facilitated by the open structure of preexisting slide materials, but the low permeability of the main slide block probably delayed mass instability until October. The impermeable shear surfaces at depth trapped water, permitting saturation and leaching of clays, buildup of pore pressures along potential slide surfaces, and buildup of a hydrostatic head and an adverse load in the preexisting graben materials. Supporting evidence of these perched zones was obtained during the repair stage of excavation.

LANDSLIDE STABILIZATION

Remedial measures can be divided into two phases: (1) winterization measures initiated in December 1978 following the October event, and (2) remedial grading and construction between January and June 1979.

Winterization measures consisted of: (1) construction of a 2-m-diameter storm drain 200 m in length in Bluebird Canyon, a project that was completed 14 hours before the first major winter storm, (2) grading of access roads to salvage cars, furniture, and other belongings of the residents and to provide access for peripheral residents and exploration equipment, (3) construction of asphalt berms and other devices to divert surface runoff away from the active landslide, (4) flattening of the head and flank scarps and sealing of cracks and fissures, and (5) installation of fences and barriers to prevent visitors and spectators from entering areas subject to further landslide hazard.

Remedial grading and construction consisted of three principal measures that achieved deep-seated stability: (1) a horseshoe-shaped shear key buttress, (2) an elongate gravity buttress in Bluebird Canyon, and (3) an elaborate network of subdrainage. A schematic block diagram (Figure 5) depicts these elements and the soldier piles installed at the head of the slide in advance of grading to help minimize failure of the backslope of the excavated shear key.

The shear key buttress consisted of a 10-celled prism of compacted fill placed in bedrock beneath the oldest rupture surface to support the upslope property. This massive fill was placed by grading machinery, chiefly bulldozers, scrapers, and compactors, operating round the clock. The 1978 slide and ancient slide materials were entirely removed from the shear key, stockpiled, and then placed in the shear key excavation and also in Bluebird Canyon as a gravity buttress. All loose slide debris resulting from backslope and sideslope failures was removed by stripping and benching.

The gravity buttress consisted of compacted fill placed in the slide-modified canyon bottom. Brush and loose surficial slide debris were cleared from the floor of the gravity buttress, but most of the 1978 slide debris and alluvium in the canyon bottom were left in place, as there was to be no rebuilding in this area. This buttress was enlarged at the end of the construction phase by 14,000 cu m of excess yardage obtained from excavation of the shear key. The gravity buttress raised the channel elevation between 6 and 7.5 m.

The subdrainage system consisted of a main subdrain and a lacework of collector perforated pipes and permeable materials placed in trenches. This system was designed to drain groundwater from the slide area to the storm drain in Bluebird Canyon on both the east and west sides of the landslide. Perched water in the ancient landslide graben was confirmed during excavation of the shear key. In addition, scattered seepages were encountered in the backslope area and along the fault and basal slide rupture surfaces, but a permanent water table was deeper. Terminal sections of the main subdrain were unperforated in order that future groundwater could not enter the fill and slide materials at the foot of the slide. Filter material was placed beneath the storm drain to provide subdrainage of the slide material not removed during emergency grading.

Sixty-six vertical soldier piles consisting of steel beams interconnected at the surface by welded grade beams were constructed upslope of the horseshoe-shaped head scarp. They were placed in 0.7-m holes and then

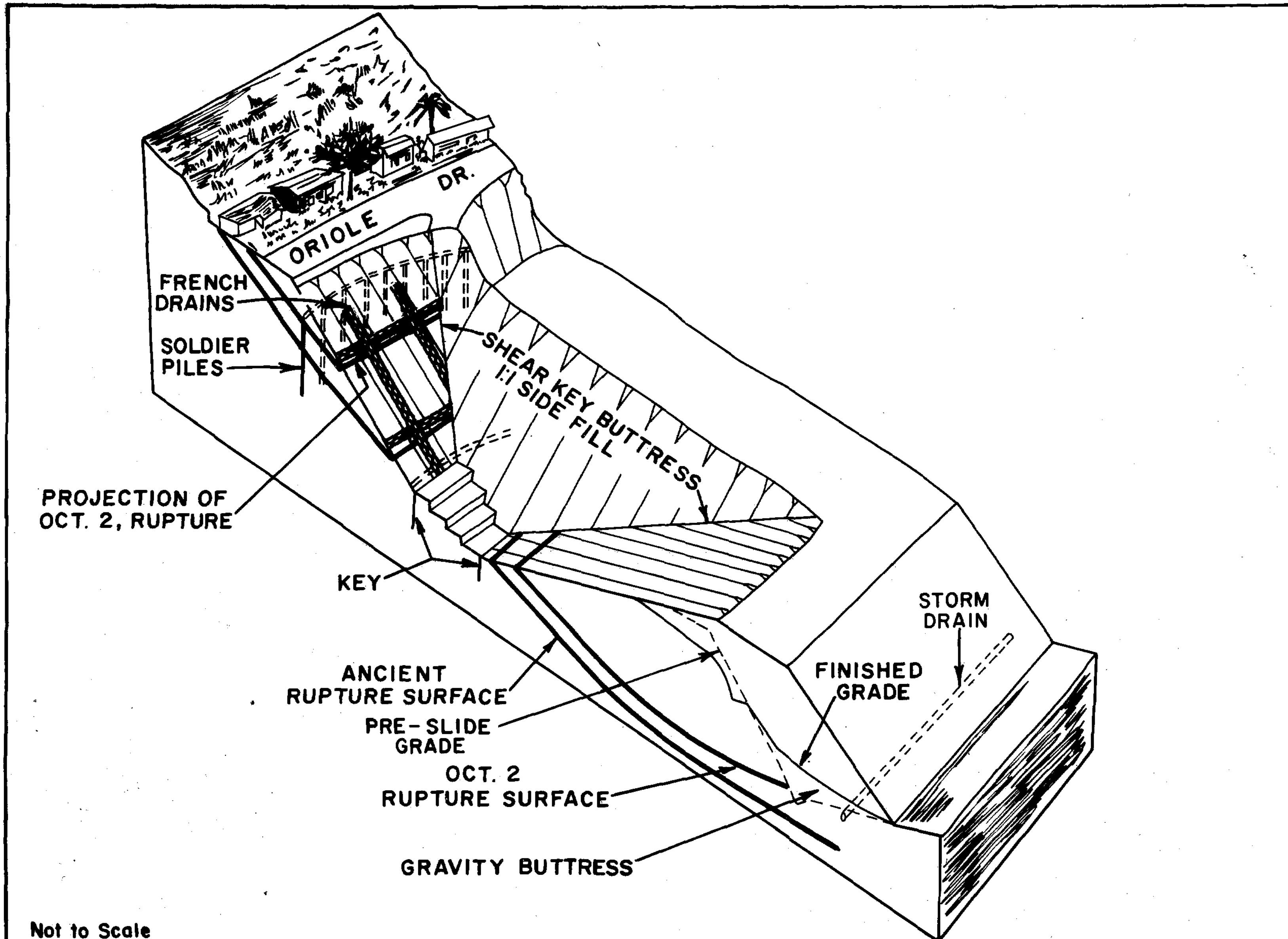


FIGURE 5 Block diagram illustrating design and construction elements.

reinforced with concrete grout. The piles reduced the number and extent of major backslope failures during the wet season. Although 16 small failures occurred during excavation of the shear key, only one produced damage. This failure bent and tilted piles, destroyed a section of street and one residence, and undermined a second residence. All other failures were downslope of the alignment of the protective emergency piles.

Geologic mapping and strain monitoring during grading were useful in predicting the location, geometry, and time of occurrence of the backslope failures. Temporary stabilization of the backslope and sideslope failures was achieved by a combination of removal, slope trimming, unloading the top of the slope, and constructing small buttresses trackwalked by bulldozers. These measures provided sufficient time to excavate the shear key buttress to design grade, install a subdrainage system, and commence the placement of the final compacted fill. As the buttress fill increased in thickness, the temporary stabilization fills were removed.

The net volume of earthwork for remedial grading was approximately 261,000 cu m. This volume does not include yardage moved a second time (60 to 75 percent of this volume) owing to physical constraints imposed by the work area available. The cost of landslide repair was about $1,920,000, compared with nearly double this amount for other alternatives. This repair was successful in saving public property and 22 residences around the periphery, in restoring 25 lots without need of further significant grading, and in preventing serious human injury. New residences have now been built upon most of these 25 lots (see Figure 1B).

REFERENCES

Leighton, F. Beach (1969a) Engineering Geologic Report of Storm Damage, City of Laguna Beach, prepared for the City of Laguna Beach, September 19, 1969, available for inspection at City of Laguna Beach.

Leighton, F. Beach (1969b) Final Geologic Report on the General Plan Study for the City of Laguna Beach, prepared for Daniel, Mann, Johnson, and Mendenhall, December 20, 1969, available for inspection at City of Laguna Beach.

Leighton, F. Beach (1979) Origin and restoration of Bluebird Canyon landslide, Laguna Beach, California (abs.), Geological Society of America Abstracts with Programs, II (7):465, with L. R. Cann and Iraj Poormand.

Leighton, F. Beach (1980) Bluebird Canyon landslide, Laguna Beach--a geomorphic threshold event, Thresholds in Geomorphology, eds. D. R. Coates and J. D. Vitek, George Allen and Unwin, pp. 387-400.

Leighton and Associates (1975) Background Data and Geotechnical Information for the Seismic Safety and Safety Element, City of Laguna Beach, prepared for the City of Laguna Beach, April 8, 1975, available for inspection at City of Laguna Beach.

Leighton and Associates (1978a) Preliminary Geotechnical Report, Bluebird Canyon Landslide, Laguna Beach, California, prepared for the City of Laguna Beach, October 14, 1978, available for inspection at City of Laguna Beach.

Leighton and Associates (1978b) Geotechnical Report of the Bluebird Canyon Landslide, Emergency Measures, Phase 2, City of Laguna Beach, California, prepared for the City of Laguna Beach, December 16, 1978, available for inspection at City of Laguna Beach.

Leighton and Associates (1979) Final Geotechnical Report of Emergency Repair Operations, Bluebird Canyon Landslide of October 1978, prepared for the City of Laguna Beach, August 24, 1979, available for inspection at City of Laguna Beach.

Miller, Russell V., and Tan, Siang S. (1979) Bluebird Canyon landslide of October 2, 1978, Laguna Beach, California, California Geology (32):5-7.

Morton, Paul K., Miller, Russell V., and Fife, Donald L. (1973) Geoenvironmental Maps of Orange County, California, Preliminary Report 15, California Division of Mines and Geology, Sacramento.

Sydnor, Robert (1979) Bluebird Canyon landslide, Laguna Beach, California, Geologic Guide of San Onofre Nuclear Generating Station and Adjacent Regions of Southern California, Guide Book 46, Pacific Section, American Association of Petroleum Geologists, Tulsa, pp. A25-A37.

Tan, Siang S. (1979) Case study: the Bluebird Canyon landslide, Laguna Beach, Orange County, Landslides in the Los Angeles Region, California, Open File Report 79-4 LA, California Division of Mines and Geology, Los Angeles District Office, Los Angeles, pp. 224-233.

Tan, Siang S., and Edgington, William J. (1976) Geology and Engineering Geologic Aspects of the Laguna Beach Quadrangle, Orange County, California, Special Report 127, California Division of Mines and Geology, Sacramento.

Vedder, J. G., Yerkes, R. F., and Schoellhamer, J. E. (1957) Geologic Map of the San Joaquin Hills-San Juan Capistrano Area, Orange County, California, United States Geological Survey Oil Map 193, scale 1:24,000, U.S. Geological Survey, Reston, Virginia.

FAILURE OF THE SAN JACINTO RIVER LEVEES NEAR SAN JACINTO, CALIFORNIA, FROM THE FLOODS OF FEBRUARY 1980

by Kenneth L. Edwards

In 1959-60 the U.S. Army Corps of Engineers and the Riverside County Flood Control District began to construct improved levees along the San Jacinto River and Bautista Creek in the vicinity of San Jacinto, California. The levees had revetment extending below the streambed to protect against scour and undercutting. But during the floodflows of February 1980, which had an estimated frequency of occurrence of once in 25 years for the combined flow, sections of the levees on both sides of the San Jacinto River and along Bautista Creek failed. The engineer team charged with investigating the failures concluded that the most probable cause of failure was undermining of bank protection. Repairs are under way to extend the toe revetments and reinforce the faces of the levees, although additional major improvements remain to be made.

The San Jacinto River originates in the San Jacinto Mountains in central Riverside County, draining the southwest slopes of those mountains. The stream is fed by a series of generally parallel streams as it works its way to the valley floor about 4 miles east of the City of Hemet, California. The stream courses about 28 miles to the mouth of the canyon at the valley floor and then traverses the valley, first in a northwesterly direction through the easterly limits of the City of San Jacinto, and then in a southwesterly direction in the vicinity of Lake Perris to Lake Elsinore, a distance of about 32 miles (for a map of the watershed see Figure 2 of the following paper by Joe Sciandrone et al.). The drainage area of the watershed after its confluence with Bautista Creek, a major tributary, is about 250 square miles. The Bautista Creek drainage area is 52 square miles. Elevations in the watershed vary from 10,805 ft at the peak of Mount San Jacinto to about 1,460 ft near the City of San Jacinto. Stream gradients vary from about 450 ft/mile in the headwaters to about 30 ft/mile along the reach controlled by levees. The San Jacinto River is an ephemeral stream that has a history of intermittent flooding during the southern California winter storm season.

A stream gage operated by the U.S. Geological Survey is located at the Cranston Bridge on the river about 5 miles upstream of the confluence with

---

Kenneth L. Edwards is Chief Engineer with the Riverside County Flood Control and Water Conservation District in Riverside, California.

Bautista Creek. This gage has provided a continuous streamflow record since 1921. The largest peak flow recorded at this gage was 45,000 cu ft/s on February 16, 1927. The peak discharge recorded on February 21, 1980, was 17,300 cu ft/s, the second largest recorded peak. However, there have been an additional five historical floods during the past 100 years estimated to have equaled or exceeded the February 21, 1980, peak discharge.

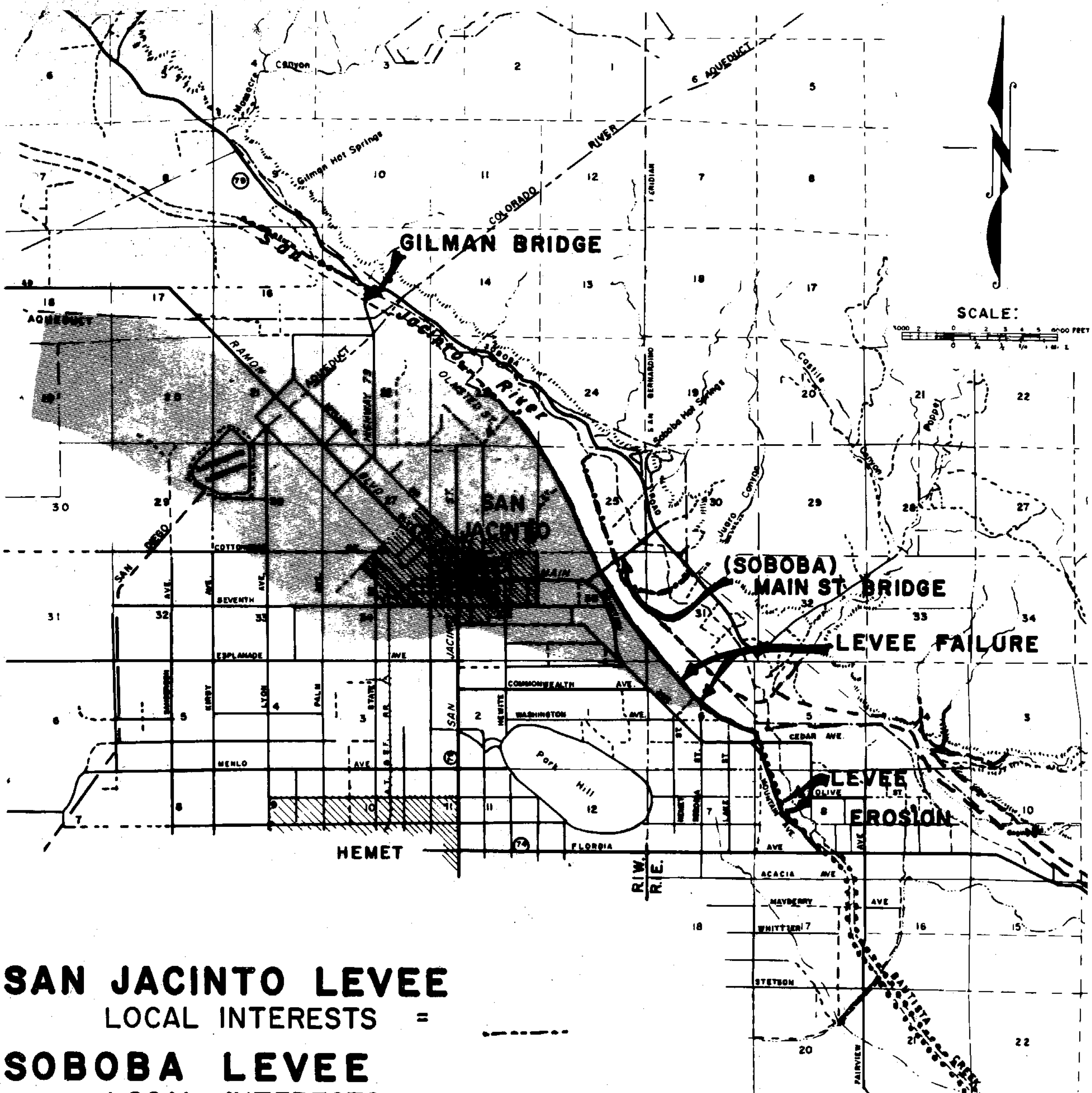
The upper reach of stream along the valley floor is an alluvial stream without an incised channel to contain even moderate flows. In the early 1900s, with the advent of agricultural development in the San Jacinto Valley, man began to erect levees to contain the river. In order to gain the most beneficial use of the land area, control efforts were made at keeping the stream in a course along the base of the mountains, where it could readily intercept the various tributary flows from the mountains. The levees were constructed from the silty, sandy soils in the streambed. Revetment, if any, consisted of pipe and wire backed with brush and cuttings from the prolific apricot groves in the valley. Because of the highly erosive nature of the soils, the steep stream gradients, and the high flow velocities, the levees failed after every flow of any significant magnitude.

The U.S. Army Corps of Engineers, in their 1946 survey report of the Santa Ana River and its tributaries, found that improvements along the San Jacinto River near San Jacinto and Bautista Creek east of Hemet were economically feasible and recommended to Congress the construction of these improvements. Funding for construction was not made available until 1959, and in 1960 work started on the Bautista Creek project. The Corps constructed a concrete-lined trapezoidal channel along Bautista Creek from near the canyon mouth downstream 3-1/4 miles to State Highway 74. A levee was constructed along the left bank of Bautista Creek from Highway 74 downstream 1-1/4 miles to the confluence with the San Jacinto River. A levee was also constructed along the left bank of the San Jacinto River from the Bautista Creek confluence downstream for 3.7 miles (see Figure 1).* The Bautista Creek channel and levee were designed for a standard project flood discharge of 16,500 cu ft/s, and the San Jacinto River levee was designed for a standard project flood discharge of 86,000 cu ft/s. The standard project flood has a frequency of occurrence of about once in 250 years.

The Bautista Creek levee was about 5 ft high above streambed, and the San Jacinto River levee was about 11 ft high. The exposed face of each levee was protected from erosion and scour by 18 in. of quarried rock placed upon a 6-in.-thick gravel filter blanket. The revetment extended about 5 ft below the streambed along Bautista Creek and about 10 ft below the streambed along the San Jacinto River. The revetment below the streambed was to protect against scour and undercutting, which could cause a displacement of the rock facing and erosion and failure of the levee (see Figure 2). The rock was placed as loose rock riprap. Gradations were such to ensure against large voids and provide interlocking of the irregular planes of the rock surfaces to

---

*See also Figure 3 of the following paper by Joe Sciandrone et al.



SAN JACINTO LEVEE
LOCAL INTERESTS =

SOBOBA LEVEE
LOCAL INTERESTS =

SAN JACINTO LEVEE
U.S. ARMY CORPS OF ENGINEERS =

BAUTISTA CREEK
U.S. ARMY CORPS OF ENGINEERS =

OVERFLOW AREA =

FIGURE 1 Flood control improvements and overflow area on the San Jacinto River and Bautista Creek.

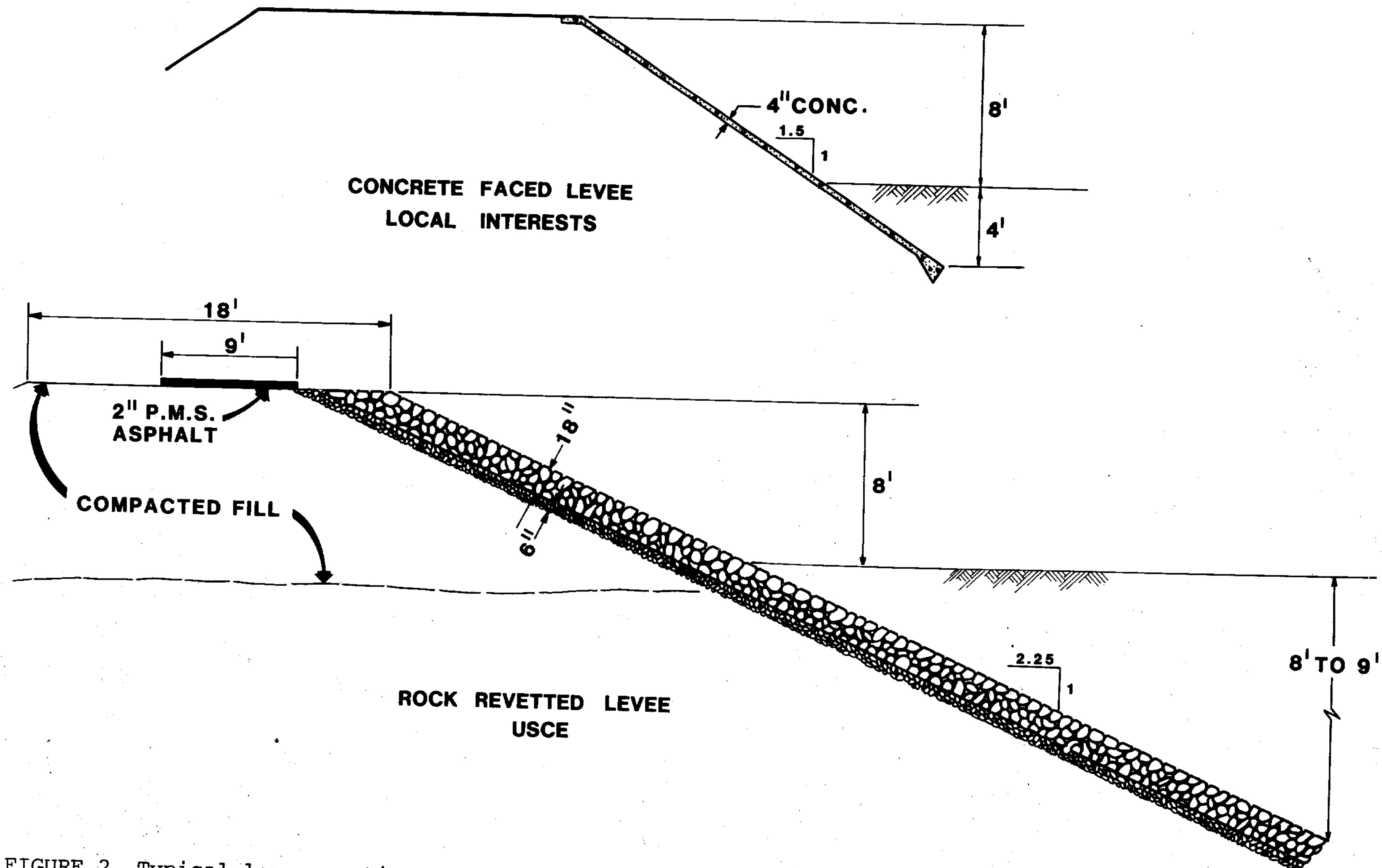


FIGURE 2 Typical levee sections.

prevent erosion of rock from the tractive and velocity forces of the river's flow against the levee face (see Figure 4 of the following paper for the size gradation).

In 1959-60 the Riverside County Flood Control District constructed a levee improvement along the right bank of the San Jacinto River downstream of the Highway 79 crossing of the river for about 1.5 miles (see Figure 1). This levee was about 8 ft high. The district elected to revet the face of the levee with a 4-in.-thick unreinforced concrete slab. The concrete slab was extended 4 ft below the streambed to provide protection from scour and undercutting of the concrete and levee toe (see Figure 2).

In 1965 the district constructed about 1.3 miles of levee along the right bank of the San Jacinto River opposite a portion of the U.S. Army Corps of Engineers levee (see Figure 1). The design and construction criteria for this reach of levee were identical to that used by the Corps of Engineers along the left bank.

The Corps' left-bank levee terminated in an open field at a point where the City of San Jacinto was protected from flooding. A low-level training dike was constructed to direct low flows back into the existing nonrevetted levees. The floods of November 1965, December 1966, and January and February 1969 breached the training dike, flooding agricultural lands and closing Highway 79. The district constructed a 5-ft-high levee in late 1969 to contain these less frequent floodflows by directing them to the river channel just upstream of the Highway 79 bridge crossing. A 4-in.-thick concrete slab was placed on the levee face. The slab extended 4 ft below the streambed to protect against scour at the toe of the levee.

Five different flood events from 1961 to 1978, with peak discharges ranging from about 6,000 to 10,000 cu ft/s, passed through the system with no damage or distress to either the Corps' or district's levees. There was sediment accumulation in the stream below Highway 79 that was removed after the 1969 and 1978 floods. The Bautista Creek channel streambed experienced significant scour during the 1965 and 1969 floods. In an effort to control the channel degradation, the district constructed a series of seven rock-filled gabion structures across the channel invert normal to the direction of flow.

A series of six Pacific winter storms passed through southern California beginning on February 13, 1980, and continuing for nine consecutive days through February 21, 1980. The last four days of the storm generally produced the largest amount of rainfall on the San Jacinto watershed. The river started flowing on February 14, 1980, and by late afternoon on February 15, 1980, flood fight operations had commenced along the left bank of the San Jacinto River downstream from Highway 79. This reach of river has only unrevetted sand levees, but very valuable agricultural land and several major highway corridors would be damaged or cut off if the river were allowed to breach these levees. On February 16, 1980, a section of concrete slab failed on the right-bank levee below Highway 79. Rock was dumped and placed to try to fill the void created at the slab failure. At 2:00 a.m. on February 17 flows were at such a volume that the flood fight had to be abandoned along the left bank downstream of Highway 79. In fact, three D-9 bulldozers were swamped and had to be abandoned at that time. Efforts continued at trying to save the right levee as more concrete sections were failing. Flows were also attacking the district's 5-ft concrete-faced levee upstream of Highway 79, and flood fight activities concentrated along that section for the next three days. Patrols and observations along the Corps of Engineers' levee indicated satisfactory performance of this levee in controlling the floodflows.

At about 7:00 a.m. on February 21, 1980, a report was called in through the Hemet Sheriff's Station that the Corps of Engineers' San Jacinto River levee was being breached. The flows in the river were attacking the levee at the breach at about a 25-degree angle (see Figure 1 of the following paper). As the flows breached they paralleled the levee for some distance, fanning out over a wide path. By the time these flows reached Main Street in San Jacinto, they moved away from the levee, flowing through the city and surrounding county areas and covering a wide floodplain for about 6 to 8 miles before joining the old stream channel and historical floodplain. By 8:30 a.m. the breach was about 700 ft wide, and it is estimated that 75 to 95 percent of the river's flow was going through the breach and through the City of San Jacinto (Figures 3 and 4). The Corps of Engineers mobilized a force of equipment and about 100 rock trucks and started hauling rock and dozing material into the breach. By about 2:00 a.m. on February 23 the breach was closed to the point of forcing flows back into the river channel. Flood fight and emergency repair activities continued for another 10 days, securing all of the levee systems to contain the remaining flows.

Bautista Creek also experienced large flows, and the left-bank levee completely failed at one location (Figure 5). Bautista Creek was degrading significantly along these reaches, and as a result the floodflows did not overtop the banks.

The flow in the San Jacinto River at the U.S. Geological Survey gage peaked at about 17,300 cu ft/s. This peak is rated at about a 1-in-30-year frequency event. The peak flow in Bautista Creek was estimated to be 6,000 cu ft/s, a 1-in-70-year frequency peak flow. The combined flow of both streams at the point of the levee breach was estimated to be 25,000 to 27,000 cu ft/s, a 1-in-25-year frequency peak flow.

The Los Angeles District of the Corps of Engineers formed an engineer team to investigate the cause of the failure and distress to their levee (Sciandrone et al., 1980). Staff members of the Riverside County Flood Control District assisted the investigation team by providing data, photographs, and other background and eyewitness accounts. A careful review was made of all design data and criteria, and aerial photographs clearly depicting the path of flows in the channel were taken. Test borings were made of the levee embankment, and trenches were excavated in the streambed around the location of the levee breach.

Several probable causes of levee failure were considered. They were:

1. Overtopping

FIGURE 3 The San Jacinto River breach (looking downstream).

2. Internal erosion (piping)
3. Slides within the levee embankment or foundation
4. Surface erosion
5. Undermining of bank protection (scour)
6. Channel configuration

After careful evaluation it was determined that the most probable cause was undermining of bank protection (Figure 6). Test trenches revealed scouring of the streambed to at least the bottom of the toe rock, after which erosion took the rock away, leaving the unprotected embankment subject to rapid erosion and failure along other reaches of the levee. There was also undercutting of the rock toe and some surface erosion of the rock facing along other reaches of the levee. The design criteria used in 1959-60 were the then-current state of the art and in accordance with prescribed Corps of Engineers criteria. Current practice calls for much thicker and heavier rock facing. The alignment of the levee and direction of flows impinging on the levee caused greater scour along the toe of the levee, thus contributing to the failure. Flow impingement at an angle (rather than parallel flow) has been shown to cause more scour; thus the alignment of the San Jacinto River levee contributed to the failure.

FIGURE 4 Overflow into the City of San Jacinto.

FIGURE 5 The Bautista Creek levee failure (looking upstream).

The failures along the district's concrete-faced levee were due to deep scour and undercutting of the toe of the concrete slab. This caused erosion and evacuation of the levee embankment under the slab and thus loss of support for the slab. The concrete then cracked and failed. The right-bank concrete-faced levee downstream from Highway 79 was nearly completely destroyed. The left-bank levee upstream from Highway 79, while never breached, nevertheless was so severely damaged that it must be replaced in total.

The Corps of Engineers is now repairing their levee throughout its entire length. They are extending the toe revetment some 10 ft deeper beyond the original toe and are grouting the entire rock face of the levee. An ungrouted rock apron 10 ft wide and 5 ft thick is being placed along the toe of the San Jacinto River levee at the location of the breach plus 500 ft upstream and downstream from the breach. Their contract for this work is $3.7 million. They are also repairing the district's levee upstream from Highway 79. A grouted stone facing will be used in lieu of a concrete slab, and an ungrouted rock apron will be placed along the toe 9 ft wide and about 3 ft thick. The rock placed on the levee during the flood fight is being used for the apron.

Additional major improvements will have to be made along Bautista Creek from Highway 74 downstream to the San Jacinto River, including improving the confluence with the river. The stone revetment along the failed or distressed reaches of the Bautista Creek levee is being extended at this time at least 5 ft deeper with a 9-ft-wide, 5-ft-thick rock apron placed in the streambed next to the levee toe for about 3,300 ft. This is considered to be an interim repair. Permanent repairs will be completed after alternative solutions can be evaluated.

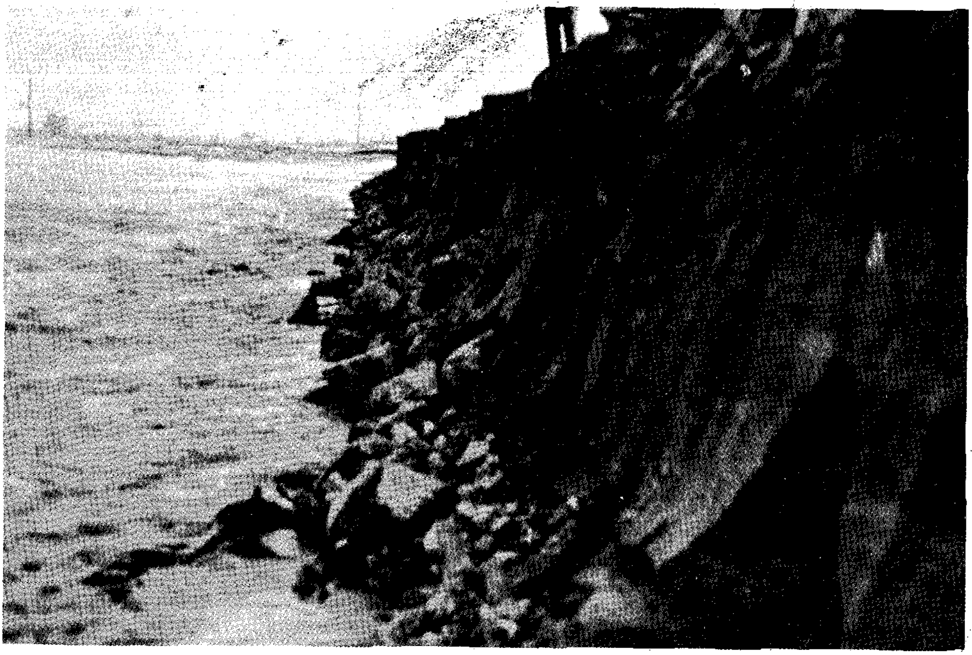


FIGURE 6 Eroded levee toe on the San Jacinto River (looking upstream).

Consideration is also being given to constructing a series of very thick heavy rock groins buried in the streambed along two different reaches of the levee where angled impingement of flows can be expected. These groins will be buried just below the streambed, will extend from the levee into the channel for about 100 ft, and will be about 12 ft thick. They would be spaced about 150 ft apart along the two critical reaches of the levee.

The failure of the San Jacinto River has taught us an important lesson. The old rule of thumb that scour depth equals flow depth is incorrect. Scour along the face of a levee can go much deeper, perhaps two to three times the flow depth. Future designs will have to consider this phenomenon.

The City of San Jacinto and surrounding areas suffered $10 to $12 million in damages (see Figure 4). Their confidence in agencies such as the Corps of Engineers and ours has been seriously damaged. It behooves those of us in this business to determine the cause of the problem and quickly rectify it. We believe we have made a significant step in that direction.

REFERENCE

Sciandrone, Joe; Albrecht, Ted, Jr.; Davidson, Richard; Douma, Jacob; Hammer, Dave; Hooppaw, Charles; and Robles, Al, Jr. (1980) Levee Failures and Distress, San Jacinto River Levee and Bautista Creek Channel, Riverside County, Santa Ana River Basin, California, U.S. Army Corps of Engineers, Los Angeles District, Los Angeles.

LEVEE FAILURES AND DISTRESS, SAN JACINTO RIVER LEVEE AND BAUTISTA CREEK CHANNEL, RIVERSIDE COUNTY, SANTA ANA RIVER BASIN, CALIFORNIA

by Joe Sciandrone, Ted Albrecht, Jr., Richard Davidson, Jacob Douma, Dave Hammer, Charles Hooppaw, and Al Robles, Jr.

The San Jacinto River levee project, located in Riverside County, consists of a 3.7-mile levee on the left side of the San Jacinto River and a 1.3-mile levee on the left side of Bautista Creek. The project is designed to protect San Jacinto, Hemet, Valle Vista, and nearby agricultural lands. At about 7 a.m. on February 21, 1980, the San Jacinto River levee was breached by a flood event estimated to be about a 25-year event. Estimates by eyewitnesses of the flow through the breach ranged from 75 to 95 percent of the river's flow.

A team of engineers was formed to determine the probable cause or causes of failure and to provide "lessons learned." The initial investigation consisted of data review and site reconnaissance, which formed the basis for recommended field investigations. Four major types of field investigations were conducted: (1) gradations of in-place riprap, (2) soil borings, (3) test trenches, and (4) scour gage recovery along the Bautista Creek reach.

The engineer team considered the following possible causes of levee failure: (1) overtopping, (2) internal erosion (piping), (3) slides within the levee embankment and/or foundation soils, (4) surface erosion,

---

Joe Sciandrone is a Consulting Geotechnical Engineer in Sacramento, California; Ted Albrecht, Jr., is Hydraulics Engineer with the U.S. Army Engineer South Pacific Division in San Francisco, California; Richard Davidson is Geotechnical Engineer with the Office of the Chief of Engineers in Washington, D.C.; Jacob Douma is a Consulting Hydraulics Engineer in Great Falls, Virginia; Dave Hammer is Chief of the Geotechnical Branch with the U.S. Army Engineer Ohio River Division in Cincinnati, Ohio; Charles Hooppaw is Chief of the Santa Ana River Resident Office with the U.S. Army Engineer District in Los Angeles, California; and Al Robles, Jr., is Chief of the Hydrology and Hydraulics Branch with the U.S. Army Engineer District in Los Angeles, California.

This paper is an abridged version of the report (Sciandrone et al., 1980) prepared by the seven authors acting as an engineering investigative team for the U.S. Army Engineer District in Los Angeles, California. It was submitted

(5) undermining of bank protection (scour), and (6) channel configuration. The team concluded that (1) undermining of the bank protection by scour appears to be the principal cause of the San Jacinto River levee failure; (2) channel configuration contributed indirectly to levee failures by producing flow impingement on levees that, in turn, produced deeper scour and undermining of the levees; and (3) although no evidence was found that surface erosion was a significant factor in levee failure, the undersized riprap protection compared with present criteria would likely be subject to failure by surface erosion during larger floods up to the design flood magnitude.

Recommendations are made for application of this experience to other projects.

INTRODUCTION

Background

During February 1980 flooding caused the San Jacinto River flood control project to undergo distress. Levees on both the San Jacinto River and Bautista Creek reaches were, in fact, breached, as evidenced in the aerial mosaic presented in Figure 1. Because of this occurrence, and at the request of the U.S. Army Engineer District in Los Angeles, an engineer team was formed and asked to determine the probable cause or causes of failure, recommend remedial construction measures, and make recommendations as to the application of this experience to existing and future projects.

---

to Norman Arno, Chief of the Engineering Division for the Los Angeles District, in August 1980. This paper was not presented orally at the symposium because the internal reviews of the report were still in progress at that time.

Due to space limitations, sections entitled "Interim Remedial Measures" and "Recommendations for Additional Remedial Measures," Appendix A ("Field Investigations of San Jacinto Levee"), and Appendix B ("Field Investigations of Bautista Creek Channel Scour Gages") have been omitted from this paper. For them see the complete report.

Dave Mann of the Riverside County Flood Control and Water Conservation District supported the team by providing data and observations from his agency. Personnel of the Los Angeles District who also supported the team include Vance Carson, Civil Design Section B, who was the team's liaison with the flood control district; Dave Cozakos, Hydraulics Section, who provided hydrologic and hydraulic information; and Terry King, Construction Operations Division, who had first-hand knowledge of flooding and emergency construction. The field investigations were conducted under the supervision of Richard Gutschow, Chief of the Materials and Investigation Section for the Foundation and Materials Branch.

A preliminary report was submitted by the engineer team following an initial investigation consisting of data review and site reconnaissance. This investigation was the basis for the field investigations conducted. This paper briefly describes the project design and construction, presents results of field investigations and findings on the cause of failures, and suggests how to apply this experience to other projects.

Project Description

The San Jacinto River levee and the Bautista Creek channel improvements are located in Riverside County. They consist of a 3.7-mile levee on the left side of San Jacinto River, a 1.3-mile levee on the left side of Bautista Creek, and a 3.25-mile concrete-lined channel on Bautista Creek upstream from State Highway 74. The federal cost of constructing this project was $3 million. The project units are designed to protect San Jacinto, Hemet, Valle Vista, and nearby agricultural areas. Since their completion in November 1961 the units have been maintained by the Riverside County Flood Control and Water Conservation District (RCFC and WCD). During the 1969 floods they prevented damages estimated at $1.3 million.

PROJECT DESIGN

The bases for design are detailed in three reports prepared by the Los Angeles District (U.S. Army Engineer District, Los Angeles, 1959a,b, 1960). The following sections contain pertinent information on the bases for design presented in these three reports.

Hydrology

Design Memorandum No. 1 (U.S. Army Corps of Engineers, 1959a) presents the hydrologic information pertaining to the design of the project. After publication of that report the project plan was changed to provide for the extension of the upstream end of the San Jacinto River levee to the downstream end of the Bautista Creek channel. A map of the project drainage area, showing the location of rain gages, appears as Figure 2.

The San Jacinto River project drainage area, which includes the Bautista Creek drainage area, comprises about 253 square miles. The drainage area lies generally on the southwest slopes of the San Jacinto Mountains. Elevations in the San Jacinto River subarea range from 10,805 ft at San Jacinto Peak to about 1,500 ft at the downstream end of the improvement. The main watercourse is fed principally by a series of generally parallel streams from the San Jacinto Mountains. The longest watercourse is about 35 miles. The gradient ranges from about 450 ft/mile in the headwaters to about 30 ft/mile near the downstream end of the improvement.

The Bautista Creek project drainage area, which comprises about 50 square miles, adjoins the San Jacinto River drainage area on the southwest. Bautista Creek enters the San Jacinto River about 4 miles east of the City of Hemet. Elevations in the area range from about 6,800 ft in the headwaters to about 1,600 ft at the confluence with the San Jacinto River. The longest

FIGURE 1 Aerial mosiac of
the San Jacinto River.

watercourse in the Bautista Creek drainage area is about 19 miles. The gradient ranges from 1,050 ft/mile in the headwaters to about 50 ft/mile near the mouth.

The standard project flood was used as the basis for design. The flood was developed in accordance with guidelines presented by the Office of the Chief of Engineers (1952a). The standard project storm, general winter type, was employed for the drainage area tributary to the San Jacinto River levees. This storm is based on the assumed occurrence of a storm equivalent to that of January 1943 transposed and centered over the area tributary to the pertinent area. The standard project storm, local type, was used for the drainage tributary to the Bautista Creek improvement. This storm is based on the assumed occurrence of a storm equivalent in magnitude to that of March 1943 transposed and centered over the area.

The resulting standard project flood peak discharges are 86,000 cu ft/s

for the San Jacinto River improvement and 16,500 cu ft/s for the Bautista Creek improvement. The standard project flood peak discharge for the San Jacinto River is about 50 percent larger than the peak discharge that occurred during the flood of record of February 1927.

Hydraulics

The hydraulic design was based on the theoretical analyses and design practices previously approved for similar projects. The design conformed to the criteria, which applied at the time, in published chapters of the engineering manual Civil Works Construction and Civil Works Engineer Bulletin No. 52-15 (Office of the Chief of Engineers, 1952b).

Design Memorandum No. 3 (U.S. Army Corps of Engineers, 1960) describes the proposed plan of improvement and functional characteristics. Levee alignment, curve data, and profiles are shown on contract drawings in file No. 172/90

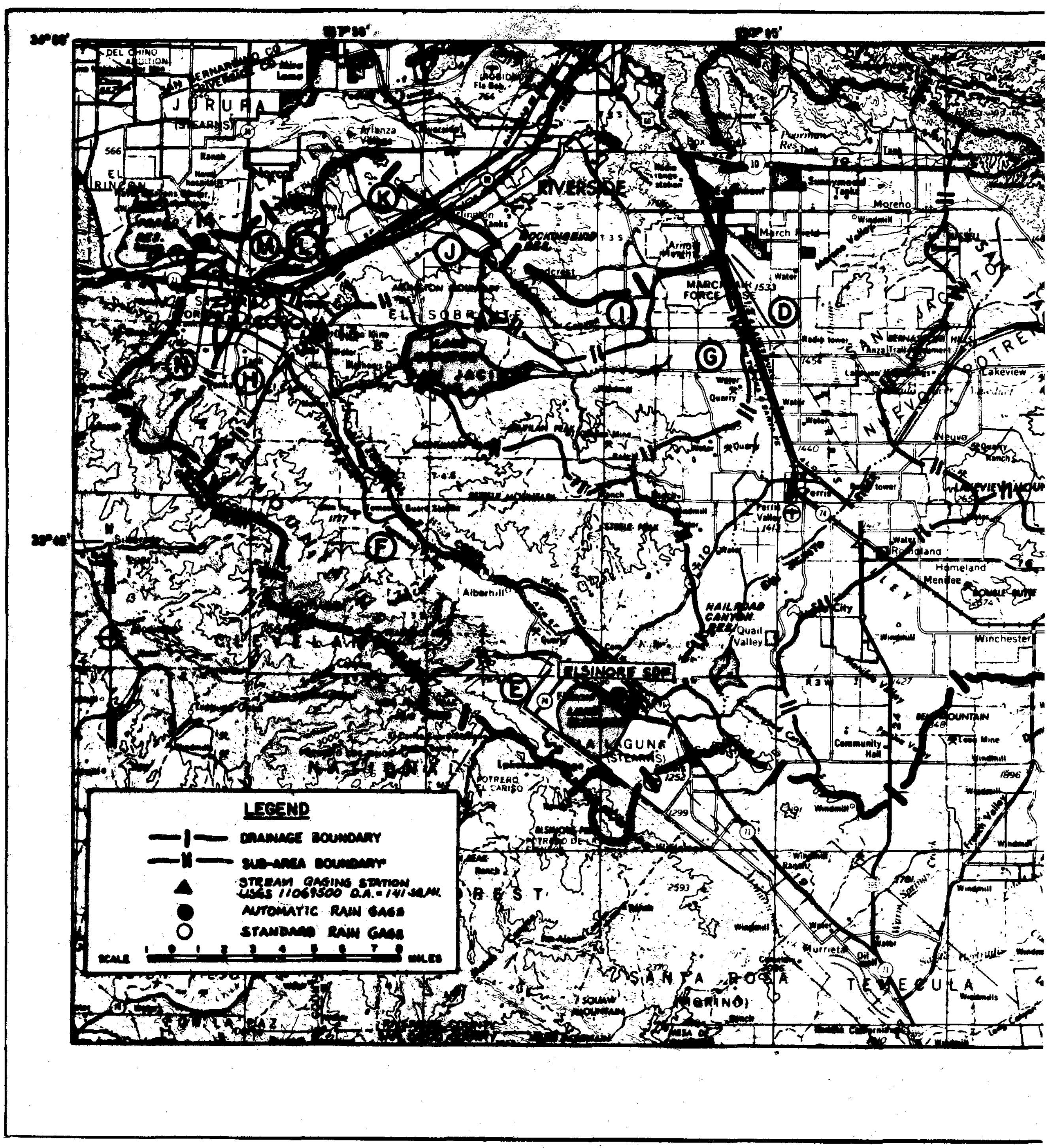
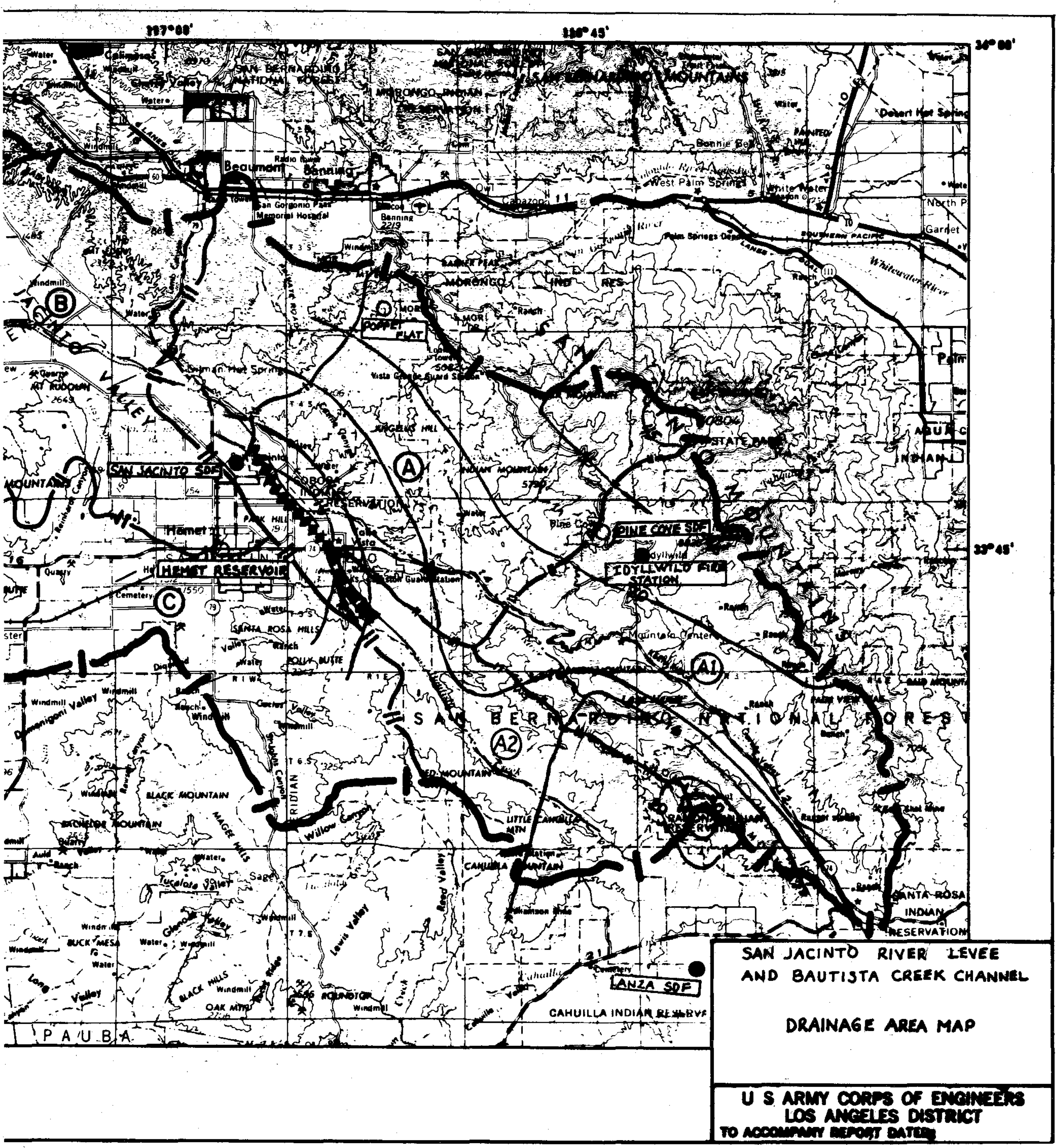


FIGURE 2 Map of the San Jacinto River and Bautista Creek drainage area.



SAN JACINTO RIVER LEVEE
AND BAUTISTA CREEK CHANNEL
DRAINAGE AREA MAP
U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED
SAN JACINTO SDF
PINE COVE SDF
IDYLLWILD FIRE STATION
HEMET RESERVOIR
ANZA SDF
POPPET FLAT
SAN BERNARDINO NATIONAL FOREST
SAN BERNARDINO MOUNTAINS
Beaumont
Banning
Hemet
Idyllwild
West Palm Springs
Desert Hot Springs
A
A1
A2
B
C

through 172/94 (D.O. Series). The preproject flood control levees of the San Jacinto River channel were constructed by local interests and were protected on the channelward side with pipe and wire fencing. The estimated channel capacity was about 8,000 to 20,000 cu ft/s, and the slope ranged from 0.00526 to 0.00935 ft/ft.

The levee along Bautista Creek was built in a reach where local interests had constructed sand levees and a pilot channel. The channelward sides were protected with pipe and wire fencing. The capacity of the preproject Bautista Creek channel was about 75 percent of the design floodflow, and the slope of the channel ranged from 0.0100 to 0.0182 ft/ft.

The water-surface computations were made by the reach method, using the Manning formula. The computations were made on the basis of a design discharge of 86,000 cu ft/s in the San Jacinto River downstream from the confluence with Bautista Creek and a design discharge of 16,500 cu ft/s in Bautista Creek. The maximum water-surface computations used to determine levee heights were based on an n value of 0.040 in the Manning formula. Depths ranging from 5.7 to 13.0 ft were computed for the San Jacinto River, and from 3.0 to 6.6 ft for Bautista Creek. The maximum mean velocities used to determine the slope and toe protection were based on an n value of 0.025. Velocities ranging from 7.3 to 15.5 ft/s were computed for the San Jacinto River, and from 9.4 to 16.9 ft/s for Bautista Creek. The water surface for the San Jacinto River was computed based on the assumption that the existing left levee would be removed and the existing right levee would remain in place. However, for Bautista Creek the water surface was computed based on the assumption that flow would be contained in an area bounded on the left by the levee and on the right by high ground.

A minimum freeboard of 3 ft above the computed water surface is provided along both streams. Superelevation was computed by the formula V^2T/gRc, where V is the velocity of flow, T is the top width of flow, g is the gravitational constant, and Rc is the radius of the curve. The superelevation of the water surface ranged from 0.2 to 1.0 ft.

Confluence computations were based on a flow of 74,000 cu ft/s in the San Jacinto River upstream from the confluence and a flow of 12,000 cu ft/s in Bautista Creek. This combination produces the maximum water-surface elevation in the confluence for the design discharge in the San Jacinto River downstream from the confluence.

Under the project document plan, the thickness of the revetment would range from 2 ft at the top of the levee to 5 ft at the toe; the revetment would be underlain by a 1-ft layer of filter material. The adopted stone revetment, a 1.5-ft layer of riprap over a 6-in. filter blanket, is shown in Figure 3. The revised thicknesses were based on the then "present-day criteria."

The depth of toe was an item of considerable concern during the design of the project, as indicated by a review of district records. The adopted depths of toe for the Bautista Creek channel and the San Jacinto River levee were 5

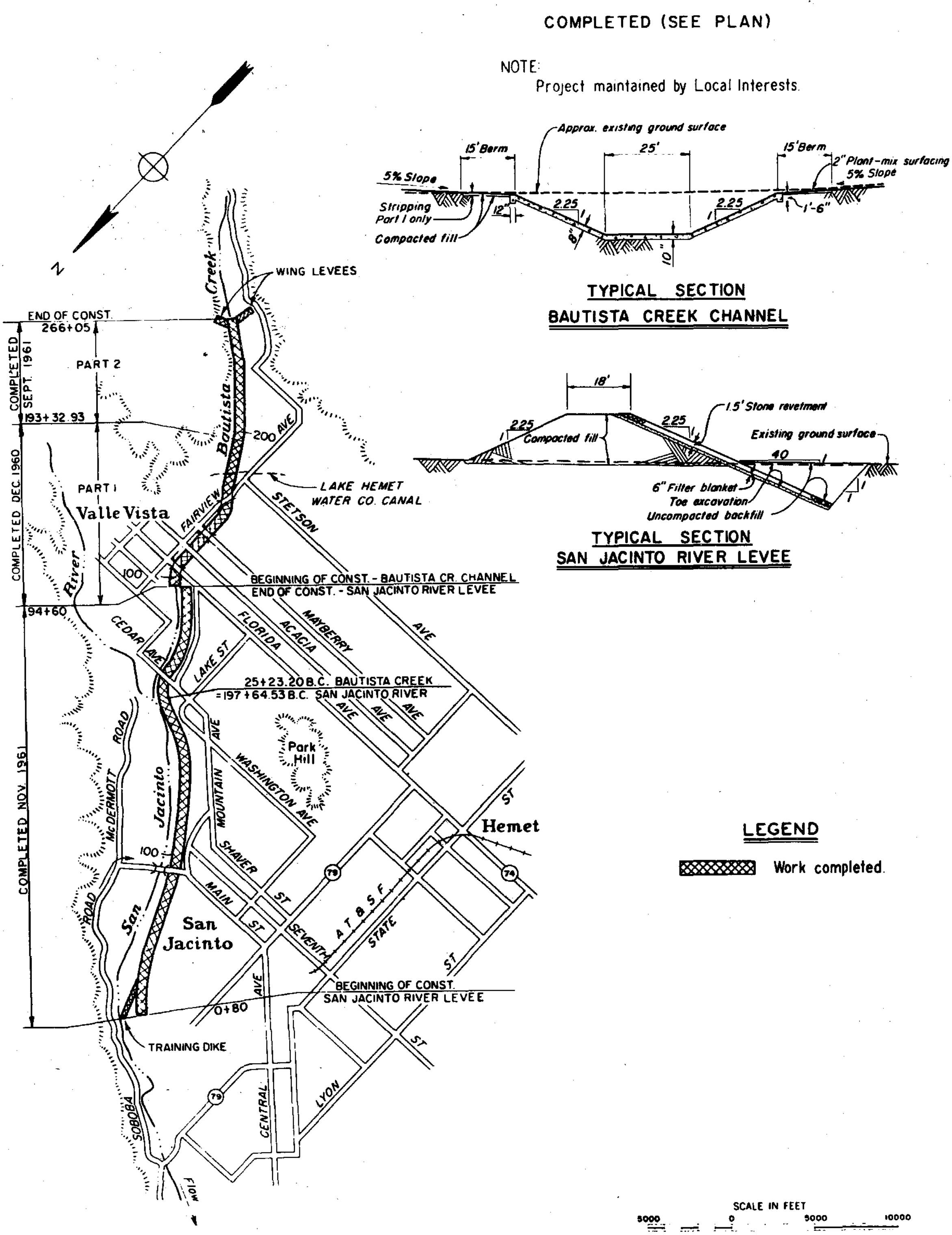


FIGURE 3 The San Jacinto River levee and Bautista Creek channel.

and 10 ft, respectively, below the low point of the streambed. Scour gages were constructed along the Bautista Creek levee.

Embankment and Foundation

The foundation materials are principally silty sands, sand-silty sands, and silts, with occasional gravel and cobbles. The upper 6 to 12 ft are loose to medium dense.

Groundwater was not found in any of the test holes that were drilled to a maximum depth of 35 ft along the project reach. The 1957 well records indicate that groundwater was about 10 ft below the streambed at the downstream end of the project levees and 60 ft below the streambed at the upstream end.

A typical embankment section is shown on the project map in Figure 3. Analysis of the slopes was based on drained strengths. Using the infinite slope method, the factor of safety for the end of the construction condition is 1.4. Steady seepage and drawdown conditions were not analyzed because the influence of seepage into the levee fills and foundations was considered to be negligible due to short-duration flows.

PROJECT CONSTRUCTION

The dates for the completion of construction of the various reaches of the San Jacinto River levee and the Bautista Creek channel are presented in Figure 3. The Bautista Creek channel project is a concrete-lined trapezoidal channel with an energy dissipator at the downstream end. The portion of the Bautista Creek channel downstream of the concrete channel is a left-bank levee with a typical section similar to that shown for the San Jacinto River levee. It was constructed as part of the San Jacinto River levee project.

Bautista Creek

The Bautista Creek levee has a maximum height of 10 ft, and the stone revetment toe is 8 to 9 ft below the line of backfill at the face of the levee. This distance corresponds to 5 ft below the low point of the streambed. The levee section was built with streambed materials and borrow that was obtained by removing an existing riverward levee. These materials were placed in 12-in. layers and compacted with four passes of a 50-ton rubber-tired roller.

San Jacinto River

The borrow for the San Jacinto River levee was obtained by removing about 4 miles of existing levee between Cedar Avenue and the downstream end of the project. The remainder of the levee fill came from streambed materials similar to the foundation materials previously described. Construction of the levee was the same as for Bautista Creek. The construction control data show that the densities varied from 96 to 106 percent of the standard maximum density of the American Association of State Highway and Transportation Officials.

Riprap

Stone for the project was obtained from the Bernasconi Pass Quarry and the Juaro Quarry. The locations of these quarries are shown on quadrangle sheets on file in the Geology Section of the Los Angeles District. The stone tested had a bulk specific gravity of 2.71 to 2.76 and an apparent specific gravity of 2.73 to 2.78.

The construction control riprap gradations are limited to the data shown in Figure 4. These gradations, which were taken at the plant located at the quarry, are not representative of the stone gradation on the levee, in part because of segregation that results from handling and placement. It has been verified that a jaw crusher was used to control the maximum size of stone, but it is not known whether a screen was used to remove the finer stone throughout the production. The stone was transported to the levee crown in end-dump trucks and then was dumped into a "skip" that was operated by crane. The skip was used to place the stone and drag the slope.

MODIFICATIONS AFTER CONSTRUCTION

San Jacinto River

The right levee in the vicinity of the Main Street (Soboba Road) crossing was constructed by the RCFC and WCD in 1965. The right levee has the same cross section as the left levee, the depth of toe revetment is the same as that of the opposite bank of the left levee, and the stone revetment specifications are the same as those for the left levee.

Bautista Creek

A 12-in. concrete-encased sanitary sewer line crosses the Bautista Creek channel at about sta 80+00. During the 1969 flood the sewer line was exposed. This experience prompted the design and construction of anchored, cable-tied, gabion stabilizers. Seven stabilizers were constructed and strengthened during the period from 1969 to 1978.

FLOOD HISTORY

Major Floods

Major floods that occurred before and after the construction of the San Jacinto River levee in 1961 are shown in Table 1.

February 1980 Flood

Rainfall occurred over the watershed for a period of nine consecutive days, from February 13 to 21, 1980. The daily precipitation for seven stations in or near the project area is summarized in Table 2. The locations of these stations are shown in Figure 1. Mean seasonal precipitation ranged from about 14 in. at San Jacinto to about 45 in. at San Jacinto Peak, averaging about 20 in. over the total area. Isohyets for the mean seasonal precipitation are also shown in Figure 2.

TABLE 1 Major Floods on the San Jacinto River

Date of Flood	Peak Discharge[a] (cu ft/s)
Before 1961	
February 1927	45,000
March 1938	14,300
January 1943	1,400[b]
After 1961	
November 1965	6,300
December 1966	5,700
January 1969	7,400
February 1969	4,100
March 1978	5,300
February 1980	17,300

[a]Above confluence with Bautista Creek at USGS gage No. 11069500, San Jacinto River near San Jacinto (above Bautista Creek).

[b]Low runoff due to extremely dry ground conditions at the beginning of the storm.

TABLE 2 Daily Rainfall on February 18-21, 1980

Station Name	Daily Rainfall (in.)			
	Feb. 18	Feb. 19	Feb. 20	Feb. 21
Anza SDF[a]	1.74	0.98	1.38	2.93
Elsinore SDF	1.96	1.27	1.36	1.43
Hemet Reservoir	0.66	0.61	1.18	1.51
Idyllwild Fire	2.63	1.36	2.05	6.63
Pine Cone SDF	n/a[b]	0.82	3.95	1.94
Poppet Flat	n/a	1.25	1.71	1.86
San Jacinto SDF	n/a	0.73	0.75	1.50

[a]SDF stations are State Department of Forestry stations.

[b]n/a = not available.

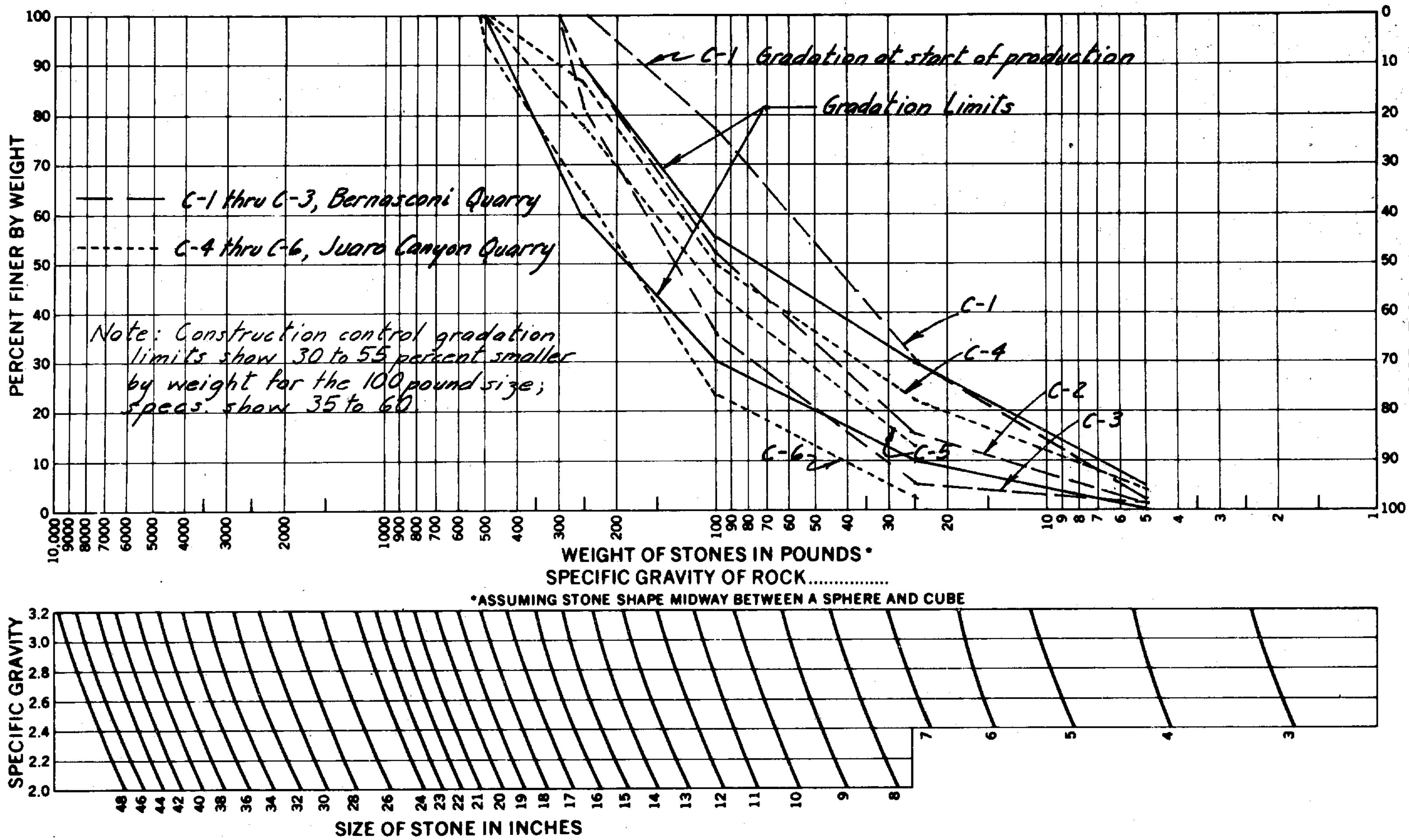


FIGURE 4 Riprap gradation curves of the San Jacinto River levee materials taken for construction control at the plant.

The peak discharge of February 21, 1980, in the San Jacinto River above Bautista Creek is 17,300 cu ft/s. Figure 5 shows the flood hydrograph. The 17,300-cu ft/s discharge represents a 30-year flood. The estimate of a 6,000-cu ft/s peak discharge on Bautista Creek represents about a 70-year flood. Based on these two discharges, the peak discharge, which occurred at the San Jacinto levee, is estimated to be about 25,000 cu ft/s, representing a flood frequency of about once in every 25 years.

PROJECT PERFORMANCE

Before the February 1980 Flood

Since the completion of the project, high flows have occurred in 1965, 1966, 1969, and 1978. In November 1965 a multiple (10) corrugated-metal pipe and dip crossing with concrete overflow at Main Street was washed out. During the February 1969 storms the Bautista Creek channel was degraded. Afterward, the seven stabilizers previously mentioned were constructed. Five of the stabilizers were damaged during the 1978 storm and were repaired in 1978 by an RCFC and WCD contract. The RCFC and WCD has kept a record of degradation and aggradation in Bautista Creek and has furnished a drawing showing streambed profiles at various times. Severe degradation of the streambed, about 10 ft, was noted before the floods of 1969. The RCFC and WCD has noted that the energy dissipator derrick stone has been repaired since the original construction.

A review of the aerial mosaics presented in Design Memorandum No. 3 (U.S. Army Corps of Engineers, 1960) and postconstruction aerial photographs indicates that topographic features have directed flows into the San Jacinto River levee in the general vicinity of the February 1980 breach. A long-time resident of the area commented after the break that it was the third time that the water broke through the same reach. The first two breaks occurred in locally constructed levees before the construction of the Corps of Engineers levee.

During the February 1980 Flood

On February 21, 1980, the Bautista Creek and San Jacinto River levees were breached. The breach in the Bautista Creek levee extended from approximately sta 61+00 to 59+00. The breach in the San Jacinto River levee extended from approximately sta 169+00 to 154+00 before flood fighting operations controlled the erosion. At several other locations erosion occurred, generally below the "line of backfill."

The RCFC and WCD has provided eyewitness accounts of the San Jacinto River levee breach. One excerpt from these eyewitness reports states: "Water Master for the Hemet-San Jacinto Area of Eastern Municipal Water District . . . was on Mountain Avenue at approximately 7:00 a.m. and observed a 20-ft-wide breach in the levee at that time and reported to their headquarters." Other eyewitness accounts following the initial breach give an account of the progress of the failure. An eyewitness account of observations at 7:45 a.m. reports: "Levee disintegrating on the upstream side of breach rapidly. Flood

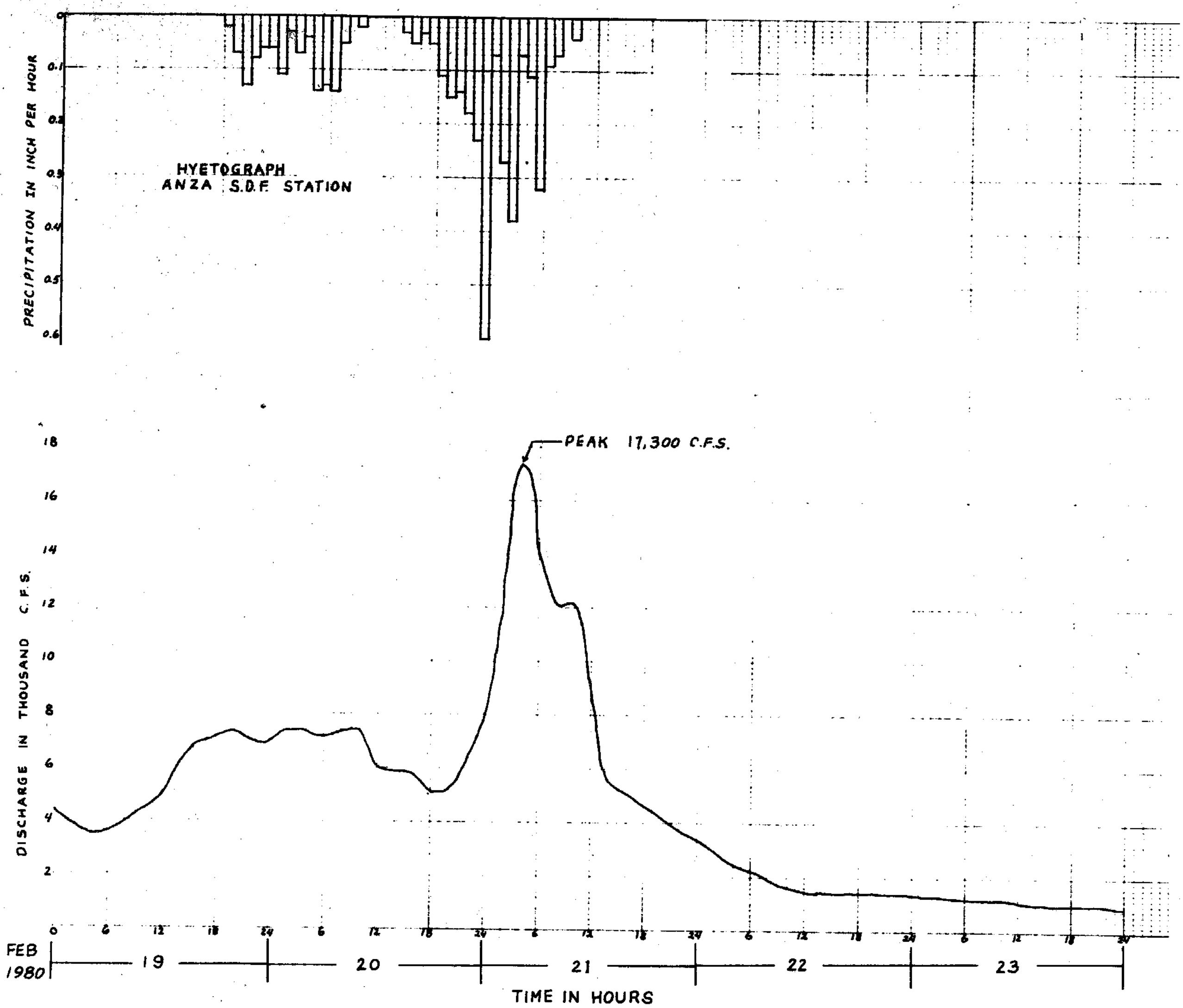


FIGURE 5 Hydrograph of the 1980 flood on the San Jacinto River near San Jacinto. Drainage area equals 141 square miles.

through breach surging in river in waves 5 to 10 ft high . . . ±8:30 a.m. Breach ±700 ft wide at this time. . . . At the location of breach the main direction of the river flow was ±25° to the downstream tangent as observed." Estimates by eyewitnesses of the flow through the breach ranged from 75 to 95 percent of the river's flow.

INVESTIGATIONS

The initial investigation consisted of data review and site reconnaissance, which formed the basis for recommended field investigations. The following sections describe the site reconnaissance and field investigations, present observations and conclusions, and evaluate results.

Site Reconnaissance

Contract drawings, topographic surveys, and aerial photographs were reviewed before site reconnaissance. The site reconnaissance itself consisted of a helicopter tour and an on-site inspection of the project. During this reconnaissance several photographs were taken of breached and distressed areas. Typical photographs are presented in Figures 6 and 7. A news photograph of the San Jacinto River levee breach (Bautista Creek) is shown in Figure 8. Observations made during reconnaissance and from photographs revealed the areas that needed field investigations.

The three areas of major damage cited on the project were (1) Bautista Creek, with one breach; (2) the San Jacinto River, with one breach; and (3) the riverside levee downstream of Main Street, which suffered extensive loss. Other areas of erosion on both projects were noted.

The existence of a ring levee around a well field near the mouth of Bautista Creek, the bar deposit at the mouth of Bautista Creek, and the upper end of the Soboba Indian levee appear to have caused flow to be directed into the San Jacinto River levee breach area. Subsequently, flow was directed into the right-side levee and then back across the streambed into the Corps of Engineers levee downstream of Main Street. As previously mentioned, damage was sustained at both areas of the Corps of Engineers levees as well as at the right-side levee where these impingements occurred.

Existing riprap is sound, exhibiting no evidence of deterioration. Areas having near-surface concentrations of smaller stone were noted. In each case where erosion of riprap was observed, it was in an area where the levee was directly attacked by flow.

There was no evidence of overtopping or of water levels even approaching the top of the levees. Minor rodent activity was observed in and near the levees. Minor erosion at the landside toe of the San Jacinto River levee upstream of the breach was noted. This feature had also been observed in the last annual inspection report.

Field Investigations

Four major types of field investigations were conducted: (1) gradations of in-place riprap, (2) soil borings, (3) test trenches, and (4) scour gage recovery. All of the above investigations were performed along the San Jacinto River levee reach except for the scour gage recovery, which was along the Bautista Creek reach. No scour gages were located along the San Jacinto River reach. Appendix A of the full report (Sciandrone et al., 1980) presents locations of the investigations along the San Jacinto River reach as well as test results. Appendix B of the full report contains results from the scour gage investigation along the Bautista Creek levee.

Riprap Gradations

Riprap gradations were performed on samples from six areas; each area was approximately 10 x 10 ft. Test areas 1 and 1A were located just upstream of

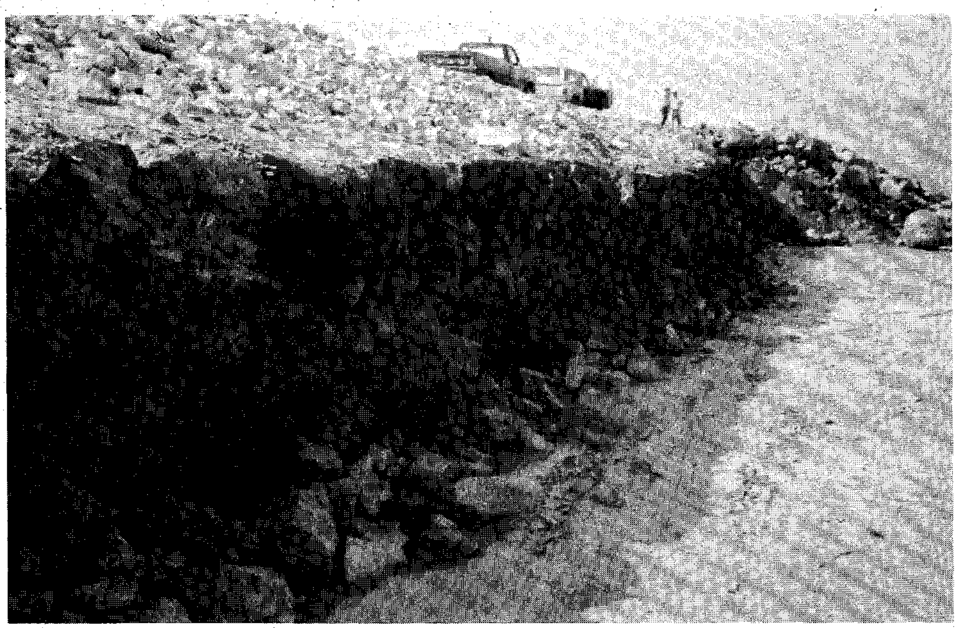


FIGURE 6 Bautista Creek levee with flood fight embankment in the background.

FIGURE 7 San Jacinto River levee in the foreground and flood fight embankment in the background.

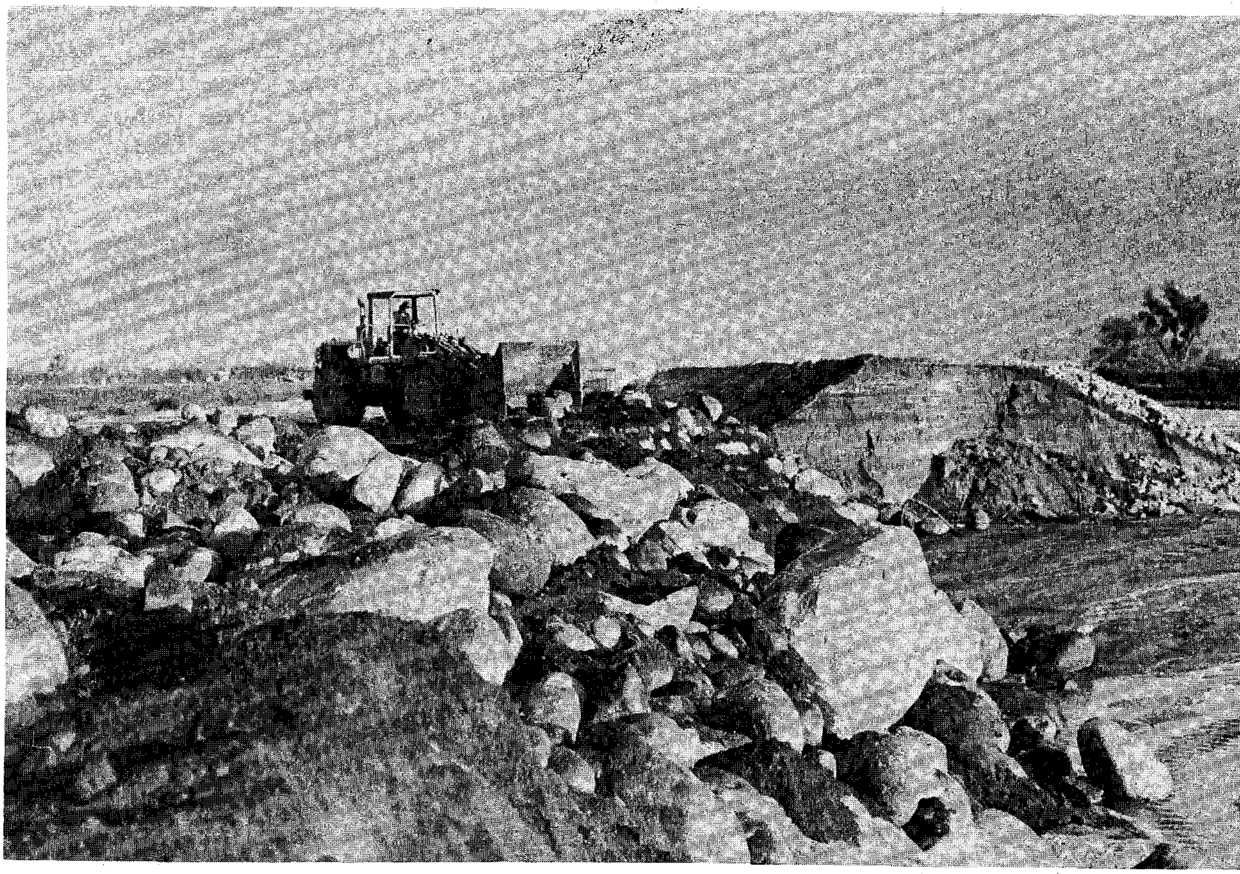


FIGURE 8 San Jacinto River levee breach at Bautista Creek. (Photograph courtesy the Hemet News.)

the San Jacinto levee breach, while 2 and 2A were just downstream of the breach. Test areas 3 and 4 were located further downstream of the breach in areas of apparently coarser-size stone and apparently finer-size stone, respectively. Results of these gradations are shown in Figures 9 and 10.

Results of the six riprap gradations made during construction for control purposes are presented in Figure 4. Results of tests 1A and 2A were judged to be erroneous after it was discovered that the scale used for determining stone weights was not calibrated properly. Therefore tests 1 and 2 were performed to replace 1A and 2A. Results of tests 1 through 4 are considered valid.

The gradation data were analyzed by determining the mean and the standard deviation for both the six construction control plant gradations and the four valid in-place gradations. The mean plus or minus one standard deviation was then used as a basis for evaluating data fit. Gradation C-1 did not fit because it was taken at the start of production, before plant adjustments were made. Gradation 4 is considered representative of the finer-size riprap. Gradations 1 and 2, taken in the vicinity of the breach, are finer than gradation 4. The in-place gradations are, on the whole, finer than the

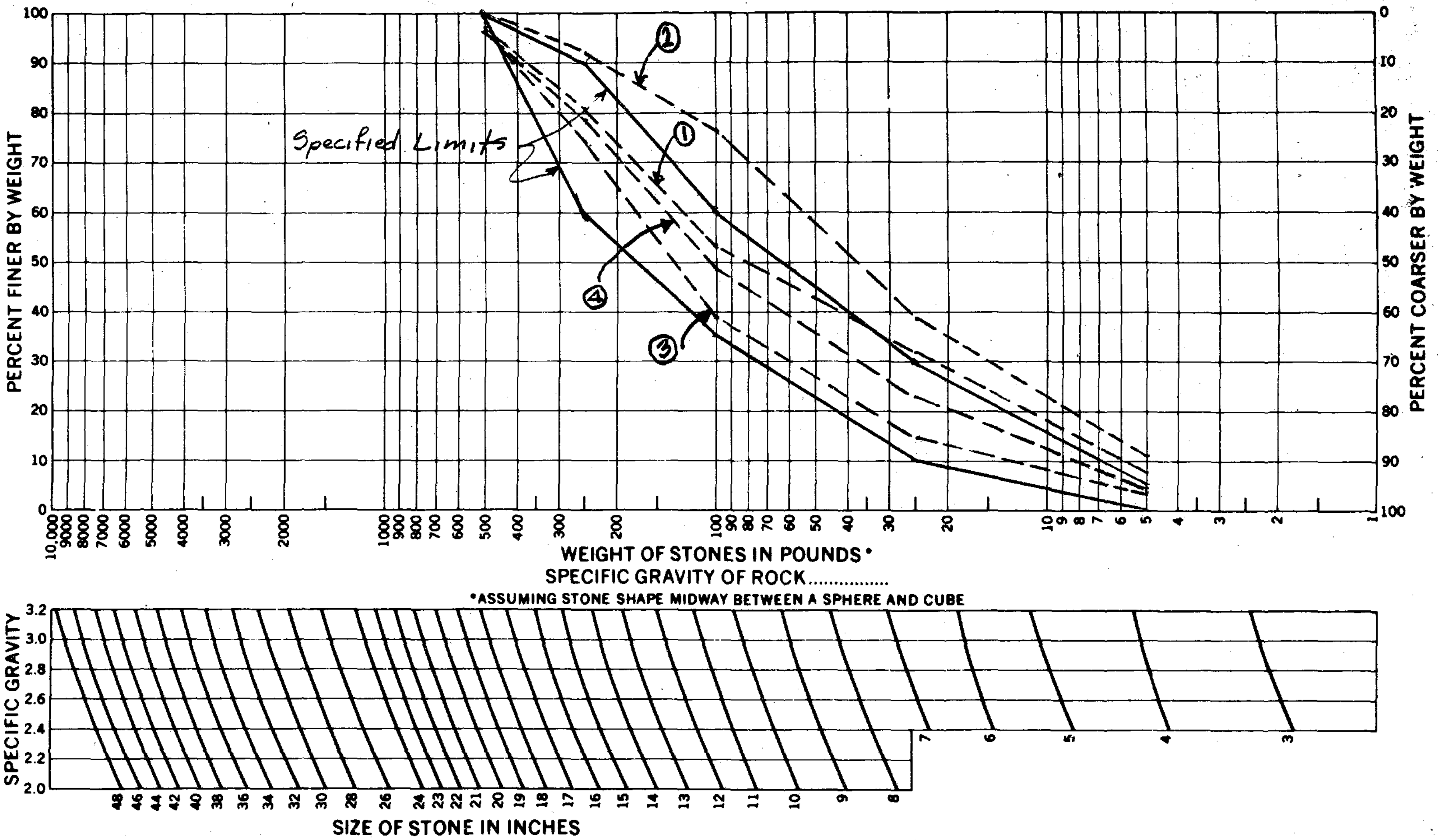


FIGURE 9 Riprap gradation curves for the San Jacinto levee. 1A: Original gradation (March 10) upstream of breach. 1: Gradation (March 26) located 10 ft upstream of 1A. 2A: Original gradation (March 10) downstream of breach. 2: Gradation (March 27) located 25 ft downstream of 2A.

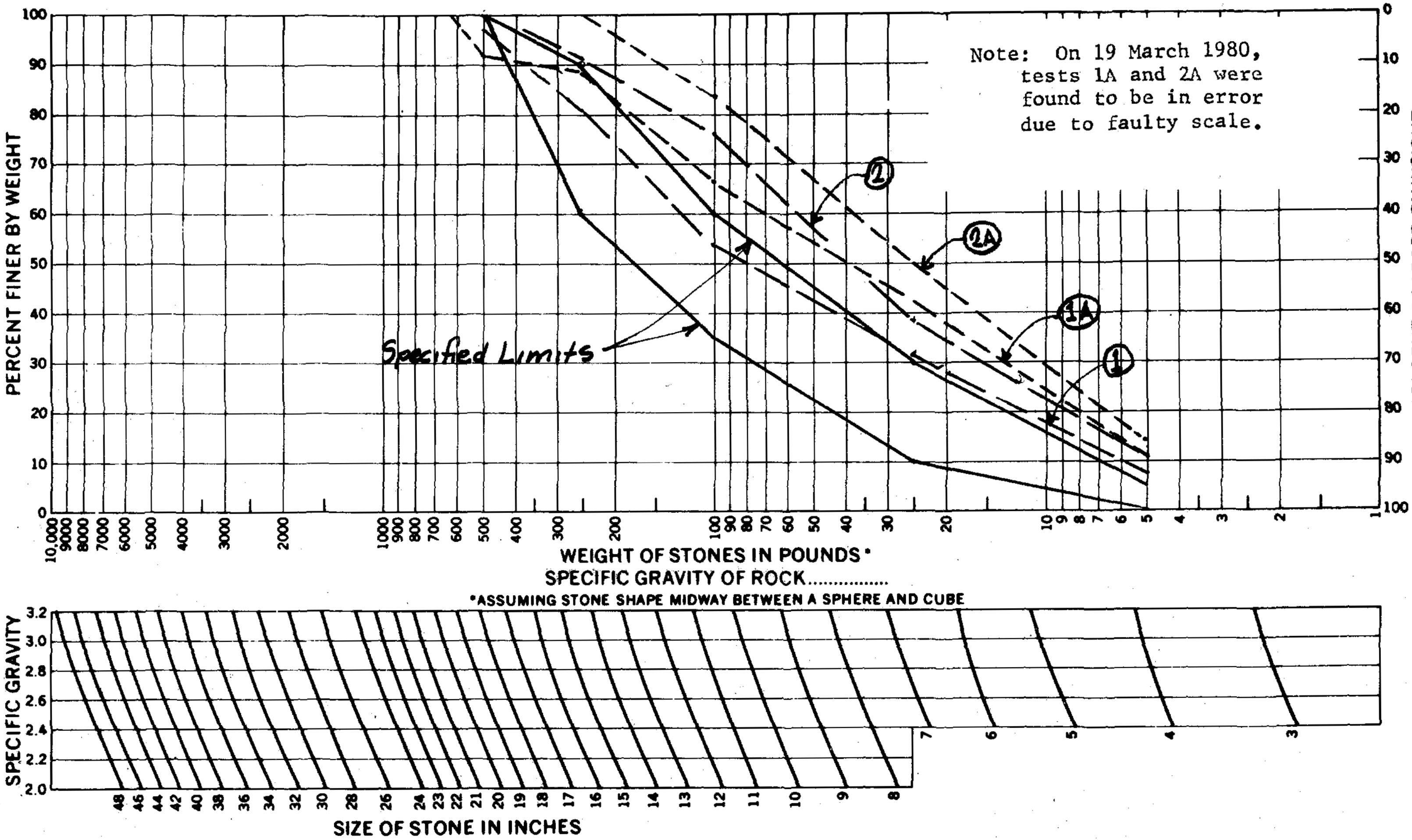


FIGURE 10 Riprap gradation curves for the San Jacinto levee. 1: Gradation taken 100 ft upstream of breach and 10 ft upstream of previous gradation (#1). 2: Gradation taken 25 ft downstream of previous gradation (located downstream of breach). 3: Taken at area visually judged to be coarse. 4: Taken at area visually judged to be fine.

construction control gradations. It was observed during sampling that some areas contain near-surface fine rock underlain by coarser rock.

Soil Borings

Five 16-in. diameter bucket auger holes were drilled during early March to determine the types and condition of embankment and foundation materials. Borings TH-1, 2, and 3 were drilled from the levee crest through the embankment and into the foundation. Total depths varied from 18 to 23 ft. Borings TH-4 and 5 were drilled to approximate depths of 35 ft in the breach area at the landside toe of the reconstructed levee. For reasons subsequently explained, three additional borings (TH-6 through TH-8) were drilled downstream of the breach during late June.

All borings indicate that the levee and its foundation consist of sands with some silts, both highly erodible materials. They also indicate that the groundwater table was at or near the original ground level during the flood. Borings TH-1 (immediately upstream of the breach) and TH-3 (approximately 275 ft downstream of the breach) had high blow counts (N values), indicating that the levee embankment consists of well-compacted material in a relatively dense state. However, boring TH-2, located approximately 200 ft downstream of the breach, had low blow counts (1 to 2) in a 7.5-ft thickness of embankment material. Blow counts of this order of magnitude denote very loose material, not at all representative of a compacted fill. Thus three additional borings (TH-6 through TH-8) were drilled to check boring TH-2. These three borings were all located on the levee crest, each approximately 5 ft from TH-2; TH-6 riverward, TH-7 upstream, and TH-8 downstream. Since no loose material was encountered in any of these borings, either the blow count from boring TH-2 was incorrect or the condition is localized. Further investigation (most likely excavation by backhoe) will be undertaken by the district, and appropriate remedial measures will be implemented as necessary

Test Trenches

Three test trenches were excavated with a hydraulic backhoe to determine the depth to which scouring occurred in relation to the levee toe. Scour depth was determined in the trenches by observing the contact between a fairly dense silty sand layer that exhibited no stratification and overlying sands that were highly stratified. The underlying unstratified silty layer was assumed to be undisturbed material that existed from the time of original construction, and the overlying stratified sands were assumed to be material deposited by streamflow; hence the contact between the two could reasonably be taken as the maximum depth of scouring.

It can be safely concluded that in both areas explored (i.e., about 1,500 ft downstream of Main Street and just downstream of the main breach) scouring could have occurred to depths at or below the levee toe.

Scour Gages

Bautista Creek is about 400 to 500 ft wide at the sections where scour gages were installed at the time of levee constructions. During installation the tops of the scour gages were set even with the channel invert existing at the time of construction. However, the survey records for the original invert elevations are unavailable. Consequently, the elevations of the two riverward gage tops had to be estimated from the original levee backfill line.

Results from the excavation and location of the scour gages are presented in Appendix B of the full report (Sciandrone et al., 1980). The deposition line given was the streambed elevation on the day of the survey, May 8, 1980. The scour line was located the same day by excavating the streambed at the gage locations until the reddish stones of the gages were encountered. The measured magnitude of the elevation difference between the initial invert and the scour line is the accumulated scour that has occurred since the time the gages were installed in 1960. These investigations indicate that (1) scour is not uniform across the creek, (2) a new gage will be needed at sta 54+58 for future measurement since only a small fraction of the first gage remains, and (3) the scour line is lower than the levee toe at sta 47+58, 54+58, and 64+58.

CAUSES OF LEVEE FAILURES

The engineer team considered the following possible causes of levee failures and their application to the subject project.

1. Overtopping
2. Internal erosion (piping)
3. Slides within the levee embankment and/or foundation
4. Surface erosion
5. Undermining of bank protection (scour)
6. Channel configuration

Overtopping

Based on high-water marks, the probable maximum height of ride-up, and the speculative height of waves and their influence on probable maximum water levels, overtopping did not occur and, therefore, was not a cause of failure.

Internal Erosion (Piping)

There was no evidence to suggest the occurrence of piping, even though the characteristics of embankment and foundation materials make them susceptible to internal erosion. Observed rodent activity is not considered to be significant. The small differential head does not produce sufficient hydraulic gradient in levee sections to develop piping. Thus internal erosion (piping) was not a cause of levee failure.

Slides Within the Levee Embankment and/or Foundation

Levee design exploration and stability analyses indicated the levee embankment and foundations to be stable. Minor erosion at the landside toe of the levee upstream of the San Jacinto River levee breach is not considered to be significant. The levee has a conservative cross section, embankment and foundation materials have high strengths, and no evidence of through or underseepage exists. Consequently, it is concluded that since slides did not occur within the levee embankment or foundation, they were not a cause of levee failure.

Surface Erosion

Levee failures can be caused by surface erosion of riprap bank protection because of action from excessive stream currents and/or waves. When riprap bank protection is subjected to currents without waves, surface erosion will occur when the tractive force produced by flow velocity exceeds the critical tractive force for stability of the stone. Waves, caused by unstable streambed formations near the bank or flow impingement on the bank (both conditions occurred in the San Jacinto River), produce uplift pressures on bank protection stone that, in combination with stream velocity, can cause surface erosion when tractive forces are smaller than the critical level. Consequently, when riprap bank protection is designed for flow velocity alone and significant waves occur along the bank, surface erosion may occur with flows substantially smaller than the design discharge.

In order to determine whether surface erosion was a cause of levee failure on the San Jacinto River, observations of in-place stone were made and four in-place gradations were taken, as previously noted. Based on visual observations, there was no evidence that significant surface erosion had occurred, although some localized areas of stone were judged to be fine and others to be coarse. The gradations, shown in Figure 10, indicate one sample to be undersized with respect to project specifications. However, the original design appears to be following the criteria used at the time of construction, namely, gradation control at the quarry only. Therefore the areas of undersized stone may be due, in part, to segregation that occurred during handling and placement.

Observations and sampling of in-place riprap indicate that, since removal of the bedding layer from beneath the riprap has not occurred, it is an unlikely cause of surface erosion leading to levee failure. Although two of the in-place gradations show the bedding layer to be finer than specified, this condition could have resulted from silting by flow sediments and/or contamination from sampling procedures, since the demarcation between bedding and embankment materials probably was not distinct. In any event it is believed that the finer gradation of the bedding material was not a significant factor in levee failure.

In trench T2, where scour depths were near the bottom of the riprap protection, some riprap was located at the scour level riverward of the riprap toe. This stone was either removed from the riprap layer by surface erosion or undermined in the breach area and transported downstream along the scoured streambed. The latter case appears to be the most likely reason for finding displaced riprap in trench T2.

Based on present criteria (Office of the Chief of Engineers, 1971), a significantly thicker layer of heavier stone would be required to withstand flood velocities (Figure 11). Although no evidence was found that surface

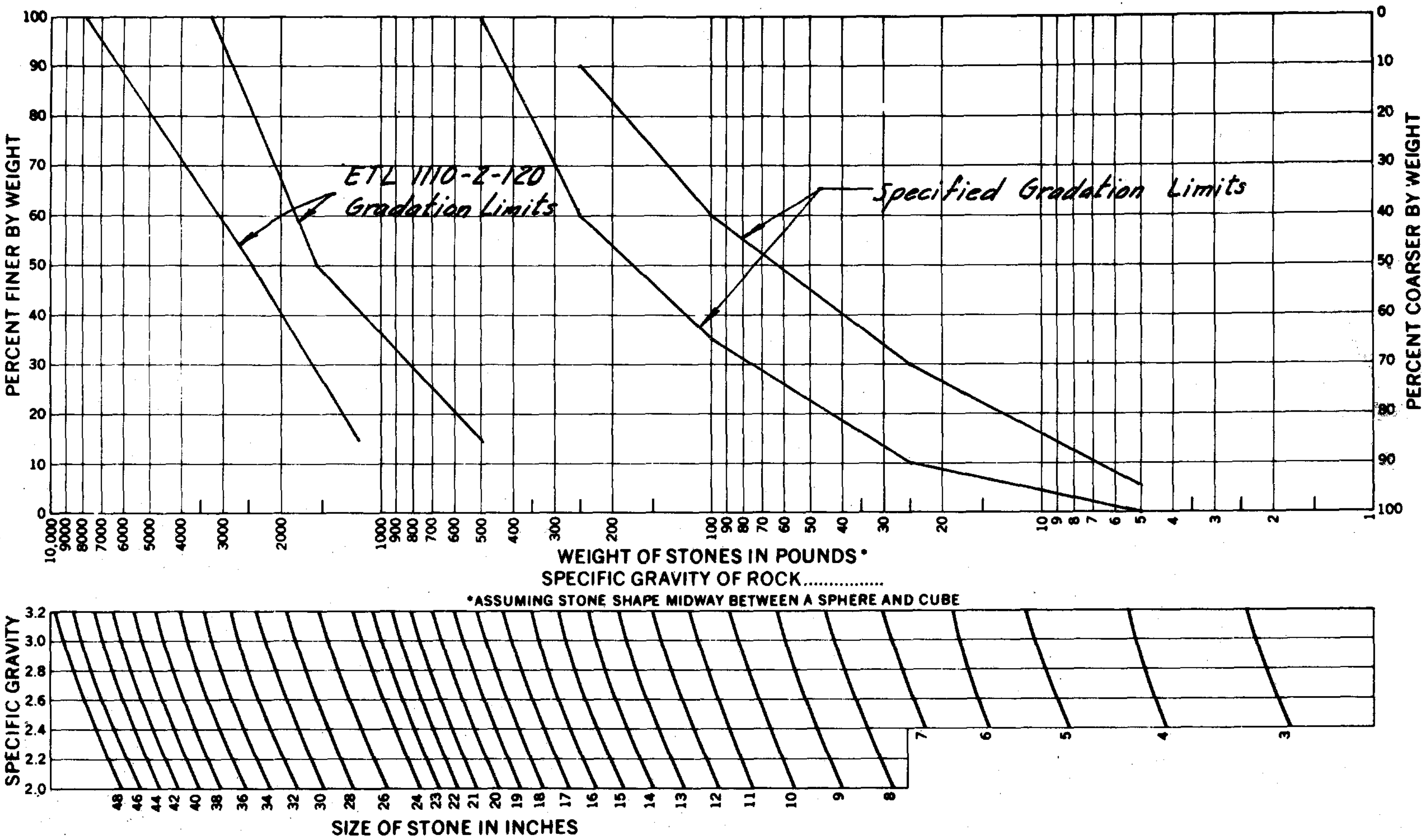


FIGURE 11 Specified versus present criteria gradation limits for the San Jacinto levee.

erosion was a significant factor in levee failure, the undersized riprap protection, compared with present criteria, would probably be subject to failure by surface erosion during larger floods up to the design flood magnitude.

Undermining of Bank Protection (Scour)

Inspection of Bautista Creek upstream of the levee suggests that construction of the concrete channel caused sediments, naturally carried by the creek, to be deposited upstream of the channel inlet. The resultant delivery of relatively sediment-free water to the leveed reach, along with the steep slope of this reach (greater than one percent), caused general streambed degradation downstream of the concrete channel. The subsequent nearly complete filling of the valley immediately upstream of the concrete channel inlet with deposited sediment and the construction of channel stabilizers by the RCFC and WCD have reduced, and in the upstream part of the reach have reversed, the general tendency of the streambed to degrade.

The RCFC and WCD has documented the general degradation of Bautista Creek through most of the leveed reach. The level of backfill (still evident along much of the levee) provides a reference plane for evaluating the approximate depth of scour and/or channel degradation. Comparing the design depth of the riprap toe with the depth of the existing streambed below the backfill reference level indicates that the streambed is at about the same level as the riprap toe along much of the levee. Visual inspection of exposed riprap at the streambed tends to confirm that the riprap toe is exposed and damaged in some locations. Examination of the scour gage data indicates that scour along the levee was approximately to the rock toe except in the breach area, where scour was several feet below the rock toe. These figures indicate that scour was 4 to 5 ft below the levee toe at sta 54+58 and 64+58, upstream and downstream of the breach. Based on observed conditions and scour gage information, it is quite evident that undermining of the bank protection caused the levee failure at Bautista Creek.

During the initial field inspection and preparation of the preliminary report, there was no readily apparent or obtainable information with which to determine the cause or causes of levee failure at the main breach in the San Jacinto River levee, other than the evidence that most of the river's flow impinged on and then flowed along the levee in the area where the breach subsequently occurred. This evidence suggested the possibility that deep scour occurred along the levee in the area of flow impingement, which undermined the levee toe and caused failure of the levee. Subsequent excavation and inspection of trenches provided positive evidence of scour depths. Trench T2, located a short distance downstream of the breach, revealed that the depth of scour was approximately to the bottom of the rock toe. Trench T3, located within the breach area and approximately 50 ft riverward of the original levee rock toe, indicated the depth of scour to be approximately at the same level as the bottom of the original rock toe. Considering the magnitude of the 1980 flood compared with other floods that occurred subsequent to completion of the project, it is reasonable to conclude that the maximum depth of postconstruction scour occurred during the 1980 flood. This evidence suggests that the maximum depth of scour at the rock toe resulting from impingement of flow on the levee face during the February 1980 flood was at or below the bottom of the rock toe at the time of the levee breach. Consequently, undermining of the bank protection by scour appears to be the principal cause of the San Jacinto levee failure.

Below the Main Street crossing the similar evidence of impingement and flow along the levee face suggests that the levee distress there was caused in the same manner as at the main breach.

Channel Configuration

The channel configuration in plan appears to have been a significant factor contributing to levee failure, inasmuch as the resulting flow impingement on the levee caused deeper scour at the toe of the rock protection. Flow impingement was particularly significant on the left levee of the San Jacinto River between sta 164+00 and 169+00. Upstream of this location the abrupt junction of Bautista Creek with the San Jacinto River and the protection wall upstream of the water well area resulted in impingement of flows at the upstream end of the right Indian levee, with some distress at that point. The upstream end of the Indian levee deflected flows across the San Jacinto River to impinge at an angle of approximately 25 degrees on the left levee at the location of failure. This angle of impingement contributed to 75 to 95 percent of the flow that passed through the levee break. Similar, but less noticeable, irregularities in the alignment of the channel bank farther downstream on the San Jacinto River and on Bautista Creek resulted in flow impingement at several locations where levee distress occurred. Thus it is evident that channel configuration contributed to levee failures by producing flow impingement on levees that, in turn, produced deeper scour and undermining of the levees.

CONCLUSIONS

Based on the information available, the engineer team has reached the following conclusions regarding the causes of levee failures.

1. Failure of the levees, in whole or in part, was caused by undermining of the levee toe, influenced by flow impingement due to adverse channel configuration.

2. There is no evidence that inadequate or improper maintenance contributed to the failure.

3. Considering the customary practices and procedures at the time of construction, the project was constructed substantially according to plans and specifications. These procedures did result, however, in riprap levee slope protection that was, at some locations, somewhat smaller than was called for in the design.

4. The riprap protection was designed based on the criteria in effect at the time. Present criteria would call for a thicker layer of heavier and more uniformly graded riprap.

5. The depth of scour was properly recognized in the original design of the levee slope protection as an important design consideration. However, the effect of flow impingement in producing greater depths of scour in certain locations was not recognized, as riprap toe protection was not taken to greater depths in those locations.

6. Two factors contributed to the failure of the Bautista Creek levee: (a) an inability to provide sufficient depth of riprap protection to accommodate the increased streambed degradation caused by a reduction in sediment load due to the presence of the upstream concrete channel and inlet, and (b) the excessively steep streambed slope in the levee reach.

APPLICATION OF EXPERIENCE TO OTHER PROJECTS

Existing Projects

Project Review

The experience with the San Jacinto River project and other experiences suggest that existing nonrectilinear channels should be reviewed to determine if conditions exist that would produce flow impingement on channel banks. Priority review and evaluation should be given to nonrectilinear, leveed, soft-bottom channels. Particular attention should be given to adverse channel alignment and to wide streams in which flows smaller than the design flow are free to meander, producing cross streamflow and levee impingement. Aerial photographs of preproject and postconstruction conditions may be useful in determining locations of adverse channel alignment, reaches of probable levee impingement, and adverse conditions at stream junctions.

The Bautista Creek experience suggests that existing nonrectilinear, leveed, soft-bottom channels on relatively steep slopes should be reviewed to determine if conditions exist that might cause excessive streambed degradation, in addition to possible flow impingement. Also, tributary streams that produce adverse flow conditions at the junction with larger streams, as does Bautista Creek, should be reviewed.

Several reports on slope protection have been prepared by the Los Angeles District. An updating and expansion of the 1971 Report on Criteria for Riprap Bank Protection, prepared by the Los Angeles District Hydraulic Section, may be used in an initial evaluation of soft-bottom channel performance. The report indicates that "layer thicknesses requirements of riprap may be larger for flows less than the maximum." It is noteworthy that damages to the Santa Maria levees in 1969 and to the Bautista Creek and San Jacinto River levees in 1980 occurred during flows that were less than maximum. As with the San Jacinto River levee, the Santa Maria levees were damaged by meandering flows that undermined the stone protection at isolated points and by cross streamflows that eroded parts of the levees.

For those reaches of levees identified for investigation and additional evaluation, in-place riprap gradation tests should be obtained. Riprap gradations taken at the plant are not representative of in-place gradations.

Current criteria (Office of the Chief of Engineers, 1970) require testing of in-place samples of riprap material

Inspection and Evaluation Program

The failure of the San Jacinto River levee signifies the need for a levee safety assurance program. An authorized program of inspection and evaluation by engineering personnel would permit reviews of soft-bottom channels that would consider current criteria, practices, and experience. The most meaningful time for such an inspection would be during periods of flow. The purpose of the program would be to identify, through data collection and review, those levees requiring early detailed investigation because of actual or suspect conditions. The detailed investigations, the determination of the need for additional defensive measures, and their construction where needed to ensure project integrity could be included under the periodic inspection and continuing evaluation program.

After evaluating existing projects and identifying locations that are likely to be damaged by design or smaller flows, defensive measures should be provided, as described below, to improve project integrity.

Future Projects

In a wide stream free to meander, the points of impingement during low water flow vary and may be indeterminate. Considering the uncertainties involved in the design and construction of bank protection, defensive measures should be provided in locations where the bank may be subject to severe angles of attack. The use of groins, as constructed on the Santa Clara levee, proposed for the Santa Maria levees, and recommended in the full report for the San Jacinto levees, should be considered, as well as deeper stone toe protection, in reaches subject to impingement. The use of channel stabilizers and/or deeper stone toe protection should be considered for channels with relatively steep slopes. Improvements in channel alignment should be made at abrupt junctions.

The construction of a rectilinear low-flow channel would channelize flow away from the bank. Considering the ephemeral nature of an excavated low-flow channel, local interests would not likely provide assurances; therefore the low-flow channel cannot be considered part of the permanent works. The low-flow channel would have to be designated as a borrow area so that local assurances for maintenance would not apply. Solutions requiring less maintenance, such as groins or deeper toe protection, are more desirable, because it seems that soft-bottom, nonrectilinear channels require more maintenance than is considered in project planning.

REFERENCES

Office of the Chief of Engineers (1952a) Project Flood Determination, Civil Works Engineer Bulletin No. 52-8, U.S. Army, Office of the Chief of Engineers, Washington, D.C.

Office of the Chief of Engineers (1952b) Slope Protection, Civil Works Engineer Bulletin No. 52-15, U.S. Army, Office of the Chief of Engineers, Washington, D.C.

Office of the Chief of Engineers (1970) Hydraulic Design of Flood Control Channels, Engineering Manual 1110-2-1601, U.S. Army, Office of the Chief of Engineers, Washington, D.C.

Office of the Chief of Engineers (1971) Additional Guidance for Riprap Channel Protection, Engineer Technical Letter 1110-2-120, U.S. Army, Office of the Chief of Engineers, Washington, D.C.

Sciandrone, Joe; Albrecht, Ted, Jr.; Davidson, Richard; Douma, Jacob; Hammer, Dave; Hooppaw, Charles; and Robles, Al, Jr. (1980) Levee Failures and Distress, San Jacinto River Levee and Bautista Creek Channel, Riverside County, Santa Ana River Basin, California, U.S. Army Corps of Engineers, Los Angeles District, Los Angeles.

U.S. Army Engineer District, Los Angeles (1959a) Hydrology for San Jacinto River and Bautista Creek Improvements, Design Memorandum No. 1, U.S. Army Corps of Engineers, Los Angeles District, Los Angeles.

U.S. Army Engineer District, Los Angeles (1959b) General Design for Bautista Creek Channel, Design Memorandum No. 2, U.S. Army Corps of Engineers, Los Angeles District, Los Angeles.

U.S. Army Engineer District, Los Angeles (1960) General Design for San Jacinto River Levee, Design Memorandum No. 3, U.S. Army Corps of Engineers, Los Angeles District, Los Angeles.

LAKE ELSINORE FLOOD DISASTER OF MARCH 1980

by Charles R. White

The severe storms in southern California throughout the early part of 1980 caused a prolonged rise in the water level of Lake Elsinore, which flooded nearly 300 homes, numerous mobile homes, and many businesses. The flooding also displaced nearly 2,000 residents. On February 19, 1980, President Carter declared Riverside County (which includes Lake Elsinore) a federal disaster area. On March 5 Governor Brown signed an Executive Emergency Order for Lake Elsinore.

Because of this flooding, staff from the Department of Water Resources (DWR) was assigned to Lake Elsinore to assist the Office of Emergency Services beginning on March 3, 1980. During the month of March DWR staff, together with the California Conservation Corps and the U.S. Army Corps of Engineers, provided flood protection for many structures in the flood-damaged area. This paper describes the situation and the preventative measures taken during this time at Lake Elsinore.

INTRODUCTION

Lake Elsinore, which is located in western Riverside County approximately 65 miles southeast of Los Angeles, occupies a terminal position in the San Jacinto drainage area. The drainage area for the river is 768 square miles (Figure 1). The existing dam on the San Jacinto River is in Railroad Canyon and has a maximum storage capacity of about 15,000 acre-ft.

Lake Elsinore is roughly rectangular in shape when the water surface is at its normal overflow elevation of 1,260 ft (Figure 2). Under that condition the lake is about 5.5 miles long from southeast to northwest, 1.8 miles wide, and has a surface area of 6,200 acres. Its average area since 1916 has been about 3,800 acres. The lake is quite shallow, having a maximum depth of only 36 ft, and has a capacity of about 130,000 acre-ft when at an elevation of 1,260 ft. The outflow of the lake goes eventually to Prado Dam and into the

---

Charles R. White is Senior Engineer with the Planning Branch of the Department of Water Resources in Los Angeles, California.

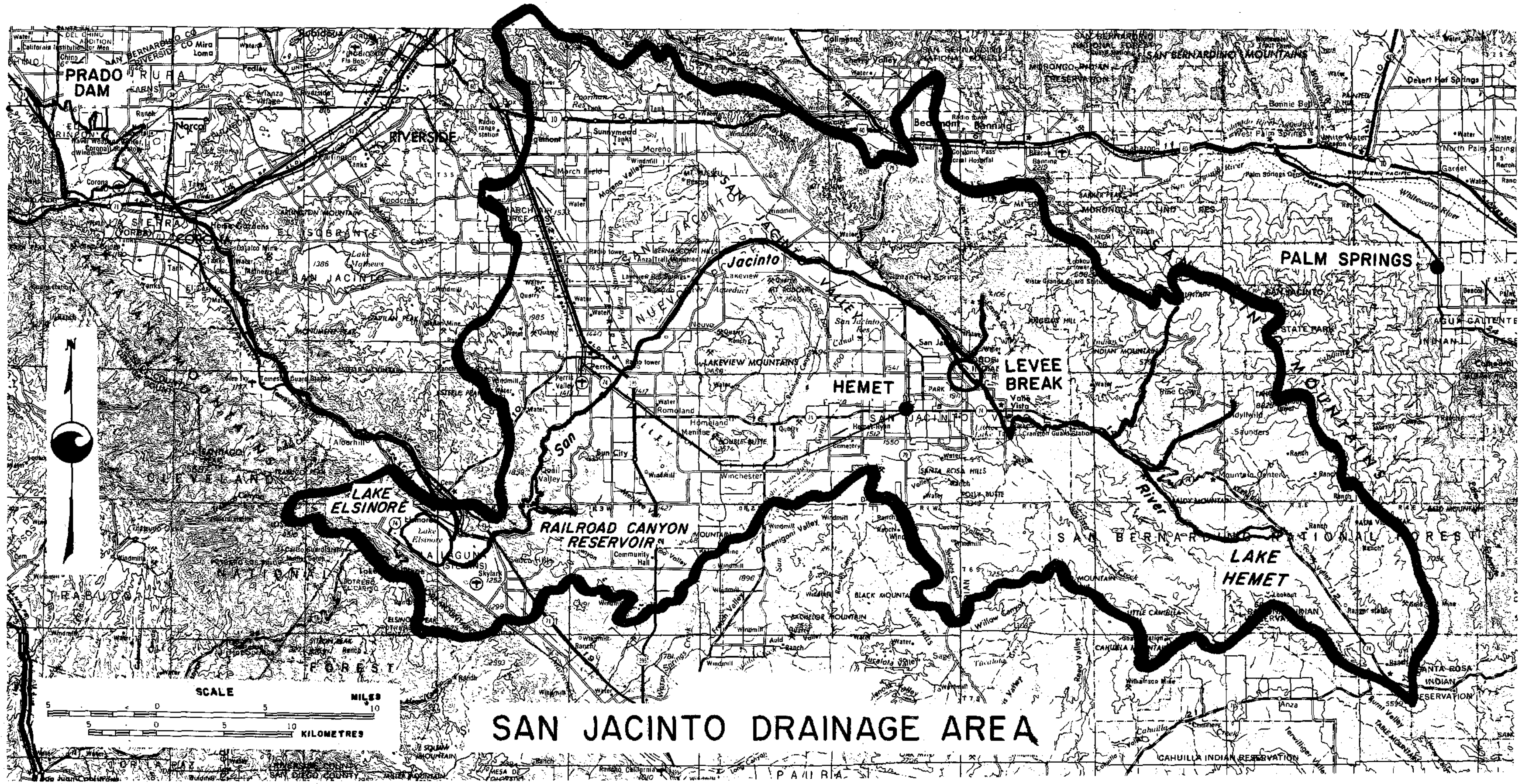


FIGURE 1 Location map of Lake Elsinore and the San Jacinto drainage area.

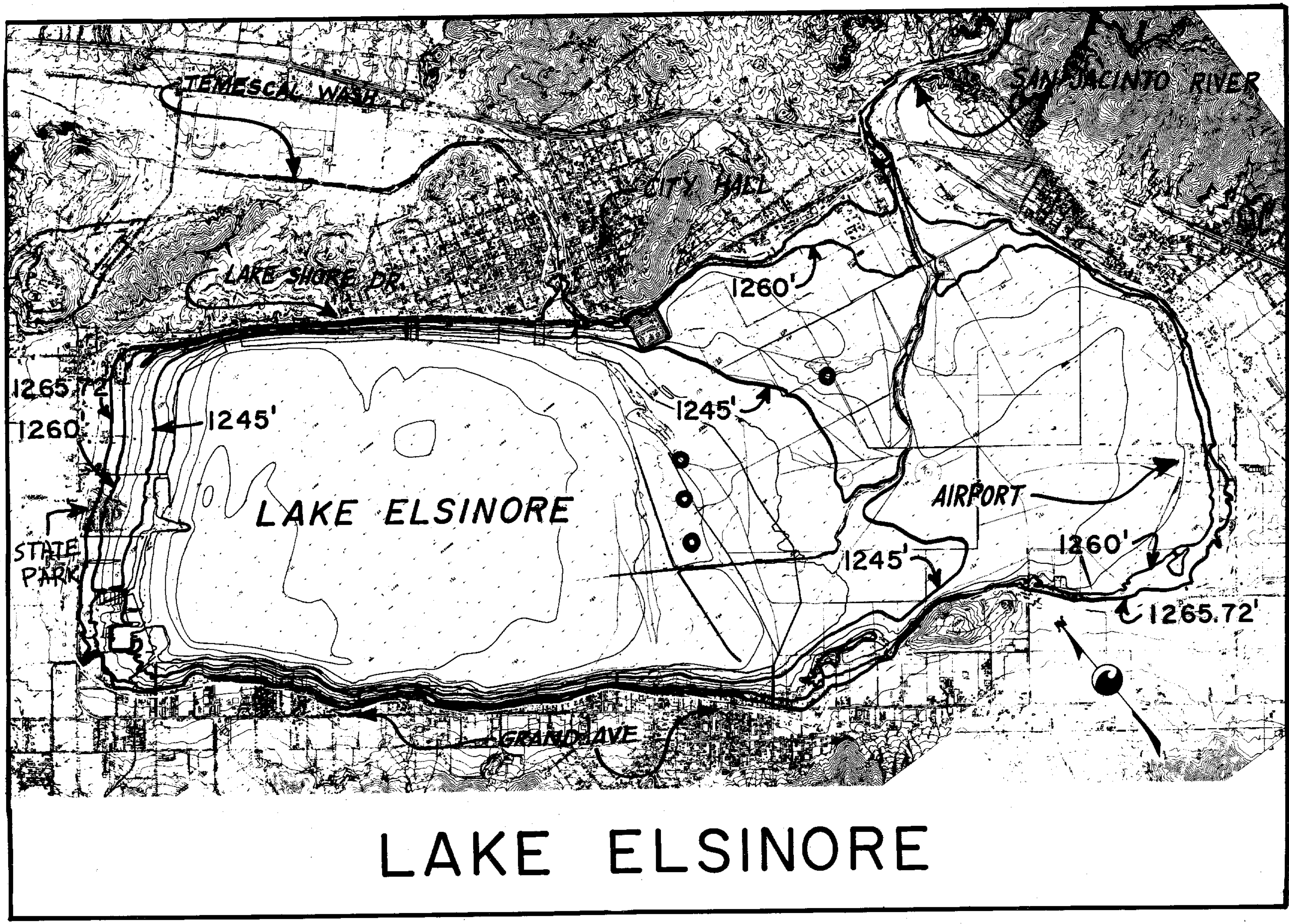


FIGURE 2 Lake Elsinore and its extent at various heights.

Santa Ana River. The outflow channel, which goes through downtown Lake Elsinore into Temescal Wash and the Santa Ana River, had not been used since 1916, the last time that the lake had overflowed.

The proximity of Lake Elsinore to the large population centers of southern California has made it a popular recreation area, particularly for water sport enthusiasts. The comparatively calm lake surface is well adapted to motorboating, water skiing, and swimming. Consequently, many homes, trailer parks, and other structures were built around the lakeshore.

In recent years the Elsinore area has attracted parachutists, glider fans, and hang glider enthusiasts. Lake Elsinore is part of a state park that has overnight campground facilities.

CLIMATE

The climate of the Elsinore area is characterized by warm dry summers and cool winters. The mean annual temperature at Elsinore is 63°F. The mean annual precipitation at Elsinore, almost all of which occurs during winter months, is about 13 in. Lake Elsinore is almost entirely surrounded by mountains. Consequently, precipitation increases sharply: the annual mean is about 25 in. only 1.5 miles from the lakeshore.

An important feature of the Lake Elsinore area is the occurrence of cycles of alternating high and low precipitation, extending for periods of considerable length. Rainfall varies between wide limits. Normal runoff into the lake is on the order of 10,000 to 15,000 acre-ft/yr.

The water level in Lake Elsinore has fluctuated depending upon the rainfall and corresponding runoff in the San Jacinto River (Figure 3). In the early 1960s the lake was dry (with an elevation of 1,223 ft). As recently as 1977 the lake was in danger of becoming dry again. The elevation was 1,228 ft. The above average rainfall in 1977-78 and 1978-79 raised the lake level to 1,246 ft by October 1979.

1980 FLOOD

During the 10-day period of February 13 to February 23, 1980, over 13 in. of rain fell. On February 13 the elevation of the water surface of the lake was 1,247 ft, with 62,000 acre-ft of water in storage (Figure 4). On February 23 the level had risen to 1,259 ft. The amount of water in storage was 124,000 acre-ft.

It was during this period in February that actions were initiated to prevent what was certain to be a disaster of major proportions. The State Office of Emergency Services (OES), the U.S. Army Corps of Engineers, and the Federal Emergency Management Agency were all called in to assist the local agencies in coping with the damages that were caused by the continued rise in the lake's water level.

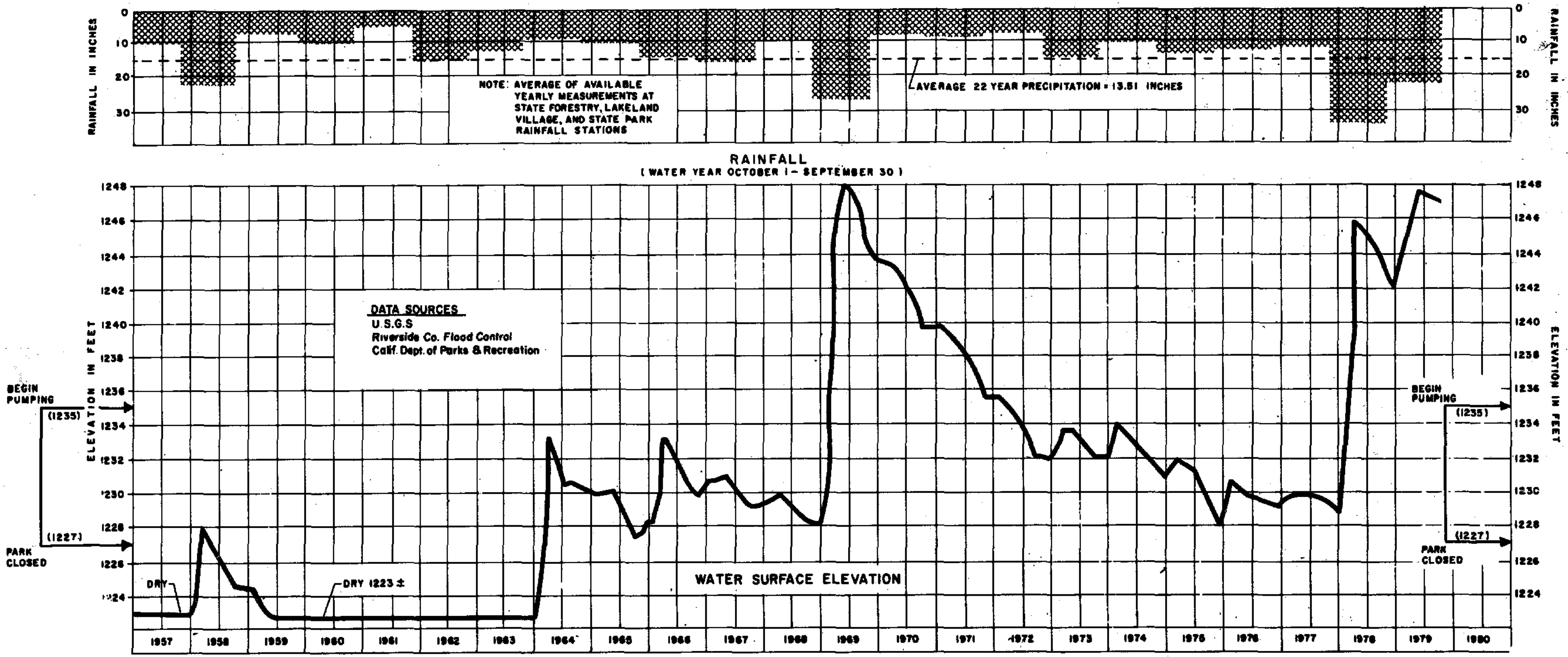


FIGURE 3 Lake Elsinore water surface elevation and rainfall prior to 1980.

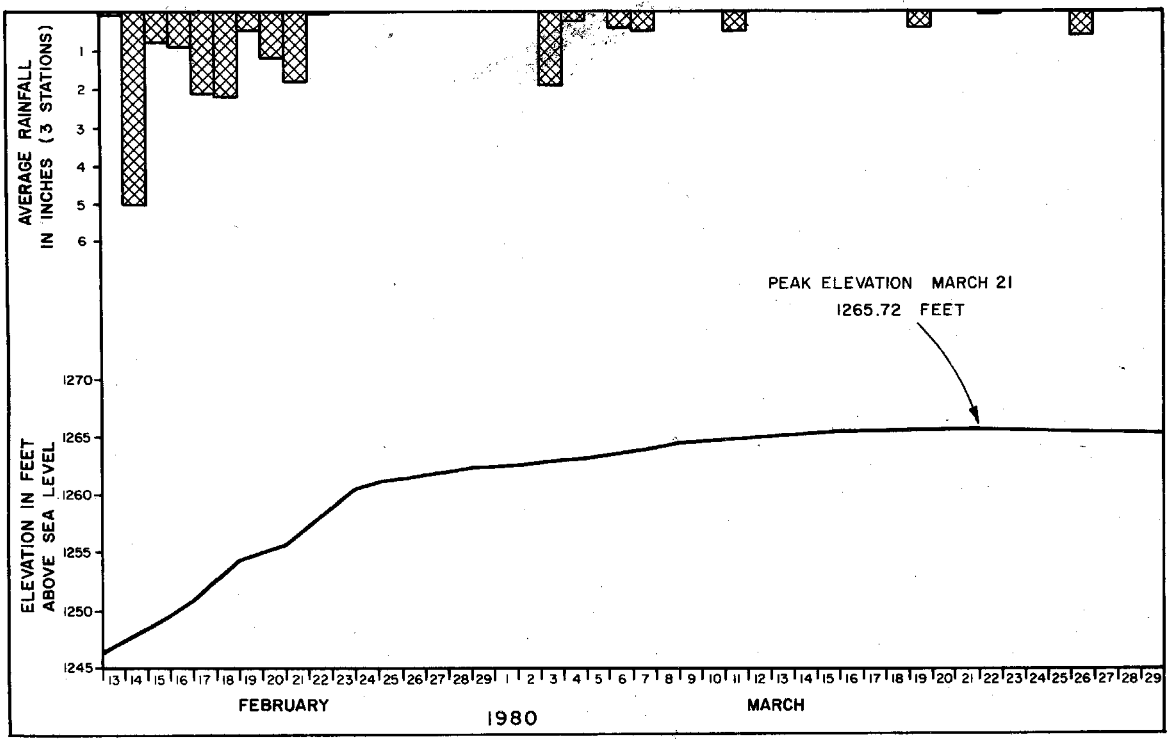


FIGURE 4 Lake Elsinore water surface elevation and rainfall.

FLOOD FIGHT

Under Public Law 84-99 the Corps initiated the dredging of the overflow channel that leads from Lake Elsinore to Temescal Wash.

Because this channel had not been used since 1916, silt and other accumulated debris had built up the elevation over the normal spill level of 1,260 ft. In some places the elevation of the channel bottom was as much as 8 ft above this. The Corps let contracts to dredge the channel to an elevation of 1,260 ft. To clear this channel required relocation of all utilities, bridges, and roads that crossed it. In effect, this cut the city into two separate parts, isolating one part of the town from the other. The Corps let the first contract on February 24.

On Sunday, March 2, the lake level was 1,262.58 ft. During the day about 1.5 in. of rain fell. The next day OES established a command post in the city. I was called upon to represent the Department of Water Resources at the command post.

Work on the channel dredging continued 24 hours a day as the level of the lake continued to rise. On March 7 the outflow channel began to allow Lake Elsinore water to flow into Temescal Wash. On that date the lake level was 1,264.05 ft (Figure 5).

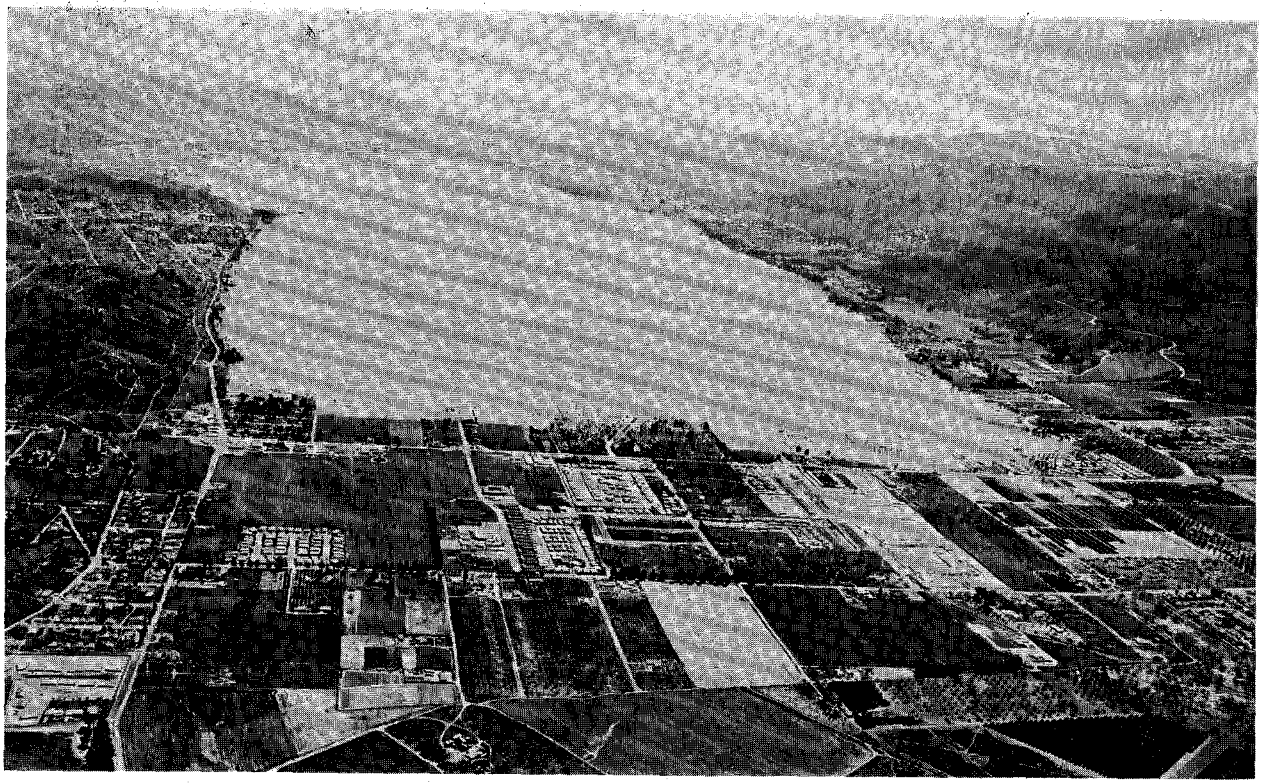


FIGURE 5 Aerial photograph of Lake Elsinore at flood stage (an elevation of 1,265 ft) in March 1980. (Photograph courtesy of Riverside County Flood Control District.)

Simultaneously with this a plan of action was being formulated to assist the residents of Lake Elsinore in coping with the continued rise in water level and resulting encroachment of water into structures along the lakeshore.

The strategy that was developed was to save as many homes, businesses, and trailer parks around the lake as possible. Already, numerous homes, businesses, mobile home parks, and other structures had been inundated by the rapid rise in the lake levels (Figures 6 and 7).

In all, three measures were taken in March (Department of Water Resources, 1980) to help alleviate the effects of the rising lake level on the residences of Lake Elsinore. One measure has already been mentioned, the dredging of the outlet channel to carry off some of the lake water.

The second measure taken was to evacuate people and property, including mobile homes that were in danger of being flooded (Figure 8).

The third step was sandbagging and the building of levees to protect homes and other developments from flooding (Figures 9 and 10).

FIGURE 6 Lakeside home under water. (Photograph courtesy of U.S. Army Corps of Engineers.)

FIGURE 7 Lakeside trailer court with lake elevation at 1,265 ft.

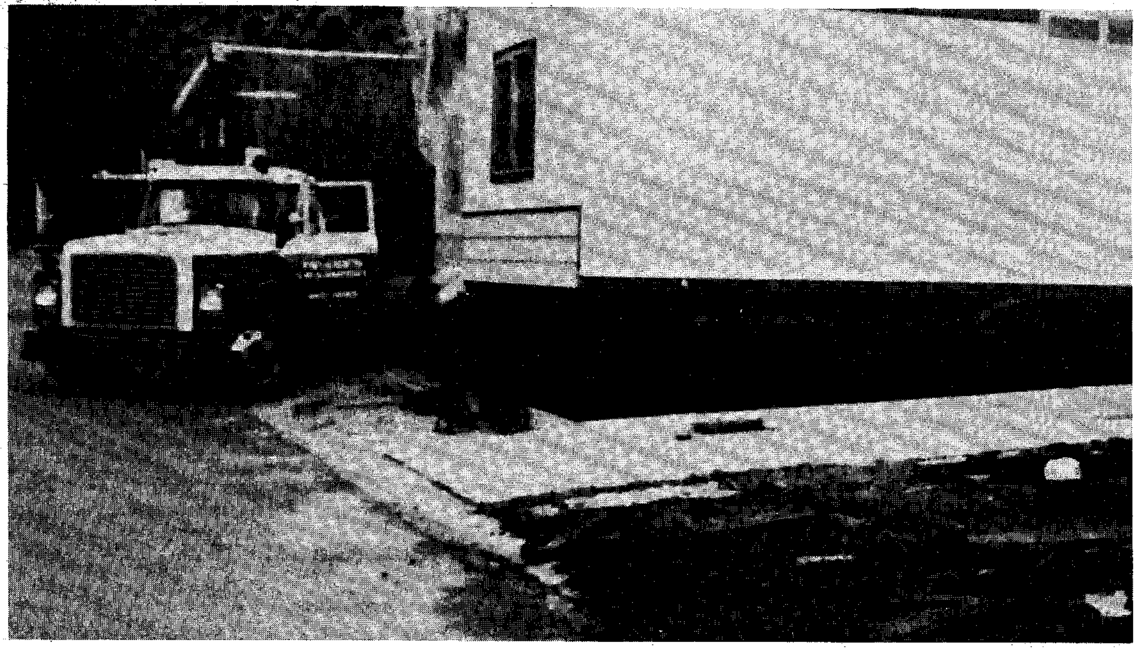


FIGURE 8 Evacuation of a double-wide mobile home at Lakeside Trailer Park.

FIGURE 9 The DWR sandbagging a home on Grand Avenue.

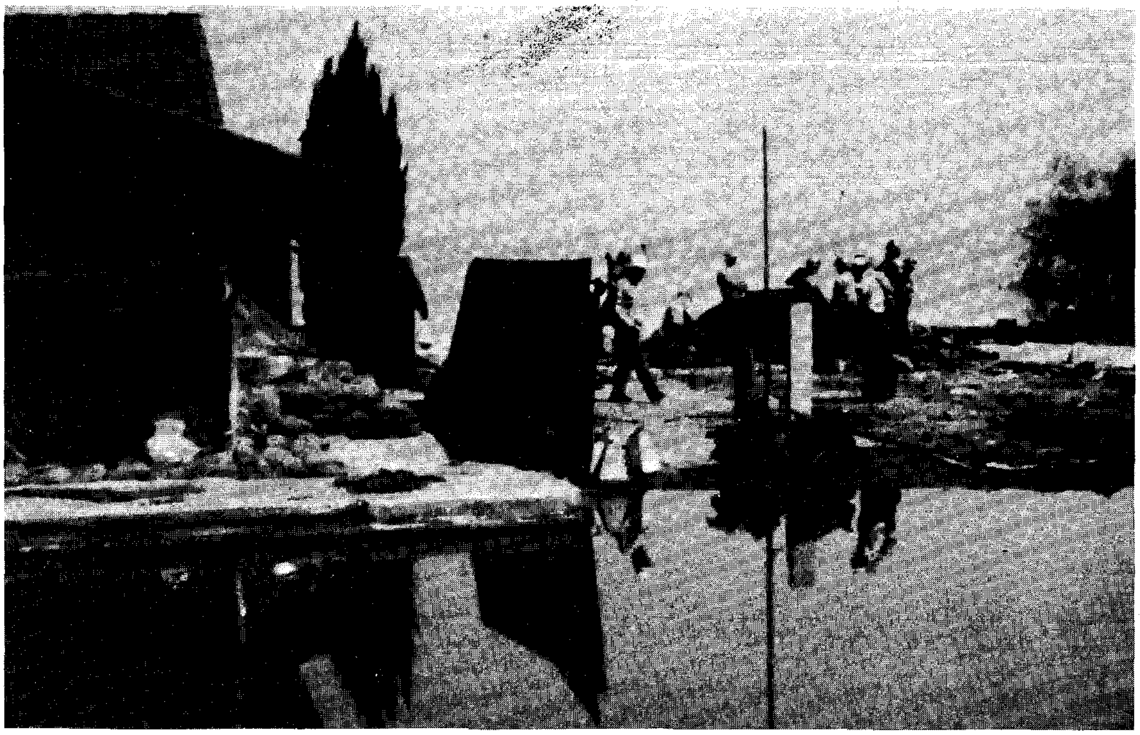


FIGURE 10 CCC crews sandbagging a home on Grand Avenue.

A flood fight situation of this type--a predictable rise of about 6 to 8 in. per day in the lake level was occurring even in the absence of new rainfall--had not occurred in recent years in southern California. We knew that time was critical. Consequently, I recommended to the OES that assistance be provided by personnel of DWR's Sacramento maintenance yard, who are experts and experienced in fighting floods.

As time was critical in the first week, an all-out effort was made to circle the lake and pinpoint the structures that were in the most danger from the rising lake levels. Each structure was visited by a DWR team of two flood fighters, then a plan of action was taken immediately with OES concurrence. Sandbags, Visqueen, and dirt were brought in and were placed in position by the California Conservation Corps (CCC) under the supervision of the DWR flood fighters. The CCC consisted of crews of young men and women whose work was outstanding during this flood.

The Corps of Engineers built four levees in Lake Elsinore (Figures 11 and 12). They were at Elsinore Village, Four Corners, Mission Trail, and Corydon Road. Coordination with the Corps was essential because all four dikes had to be sandbagged and protected with Visqueen to prevent damage from wave action and other forces. These levees were sandbagged by the CCC under the supervision of DWR flood fighters.

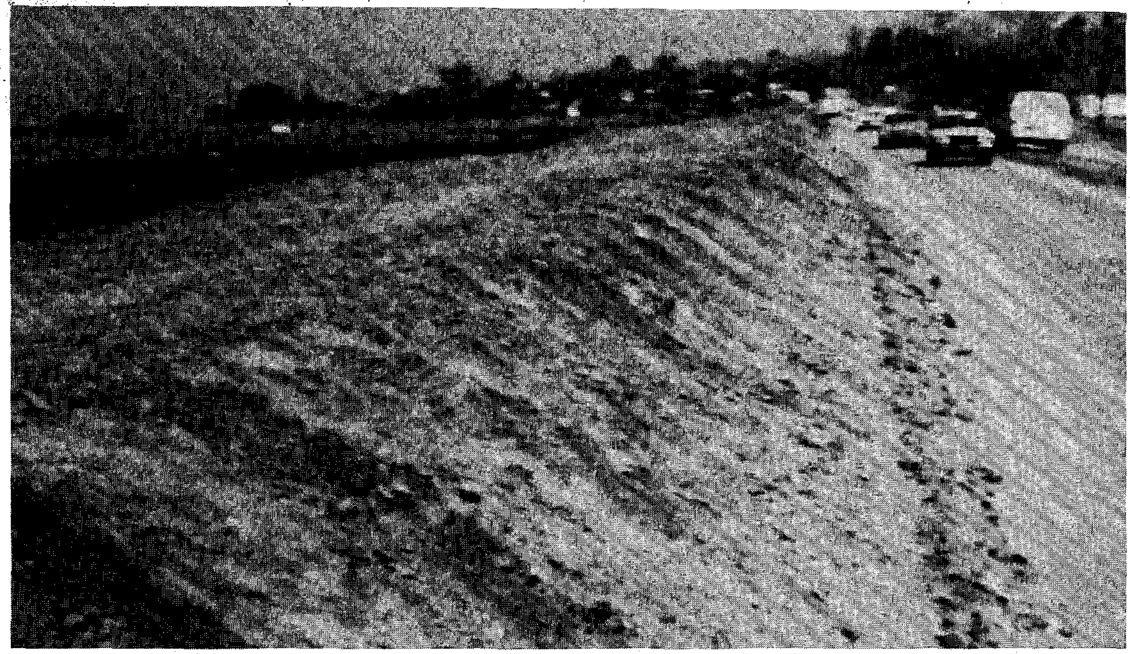


FIGURE 11 Levee at Mission Trail Road.

The above actions were taking place simultaneously; the dikes were being constructed and sandbagged and homes and other structures were being protected. In all, over 400 members of the CCC were used in fighting the Elsinore flood. We worked 16 hours a day, 7 days a week during this emergency.

Because of the relatively slow rise of the lake in March and the hard work and dedication of all public and private agencies, the damages to property in Lake Elsinore were minimized. Consider that if no outflow channel had been dredged the lake would have risen an estimated additional 1.5 to 2 ft in height. A number of homes were saved from serious flood damage by the building of levees and the sandbagging operation.

Many steps remain to be taken to prevent damages at Lake Elsinore from the rains this fall and winter. A task force of all public agencies involved in the flood fight (federal, state, and local) was formed. The purpose of the task force is to advise the governor of necessary steps and actions, including relocations to prevent and mitigate future flooding and loss or damage to public and private property in the vicinity of Lake Elsinore. A subcommittee of this task force has recommended a pumpout scheme to get the lake down below its current elevation--1,260 ft. The pumpout scheme, if all goes well, will lower the lake to an elevation of 1,255 ft, thus allowing space for water from new rains this coming winter. It is hoped that the pumping can begin by October 15.

Estimates of flood damages at Lake Elsinore are on the order of $50 million--over $40 million damage to private property and about $9 million to

FIGURE 12 Lake Elsinore levees.

public property. Without the measures taken during February and March by federal, state, local, and private agencies and contractors, these damages could have been much greater. The cooperation and teamwork shown in the Lake Elsinore flood fight are to be commended.

REFERENCES

Bennett, Lorraine (1980) "Disaster Still Lays at Shore of Flooded Lake Elsinore," Los Angeles Times, August 24.

Department of Water Resources (1980) California Flood Management, An Evaluation of Flood Damage Prevention Programs, Bulletin No. 199, Department of Water Resources, Sacramento.

Edaw, Inc. (1974) Lake Stabilization and Land Use Plan, Lake Elsinore Draft Report, Edaw, Inc., Newport Beach, California.

Lake Elsinore Emergency Task Force (1980) Subcommittee Report on Pumpout Scheme, Department of Water Resources, Southern District, Los Angeles.

State Water Resources Board (1953) Elsinore Basin Investigation, Bulletin No. 9, State Water Resources Board, Sacramento.

FLOOD CONTROL WORKS ON THE SALT RIVER IN PHOENIX, ARIZONA

by Ramsey M. McDermid, George J. Geiser,
Daryl B. Simons, and Ruh-Ming Li

The Salt River near Phoenix, Arizona, has been subjected to repeated floods over the past few years. The Interstate 10 bridge across the river is a vital transportation link for the metropolitan area. Local scour, general bed degradation, and lateral channel migration during successive flows have altered the riverbed over and around the piers of the bridge. Floods in 1978 and 1979 caused the settling of Pier No. 11. The Arizona Department of Transportation concluded that the bridge may be susceptible to further damage in the future. A hydraulic and erosion analysis for various flood control facility design alternatives, including the as-is condition and short-term channelization plans, was conducted. A 300-ft-wide low-flow channelization plan was adopted and implemented for short-term protection. In February 1980 a flood comparable in size with the design flood passed through the channel. This paper presents analyses of the susceptibility of the pier foundations to scouring during future floods.

The degradation and aggradation problem is very complicated. Simplifying assumptions are needed to obtain practical and economical solutions. The dominant physical processes include water runoff, sediment transport, sediment routing, degradation, aggradation, breaking and forming of the armor layer, etc. These processes are unsteady in nature. In order to simplify the solution and to make the results of the analysis compatible with the HEC-2 flood profile analysis, a known discharge assumption is used. The known discharge solution is appropriate in this study because of the short distances involved in the analysis. In addition, to save computer time the degradation and aggradation analysis is conducted on a reach basis using the average hydraulic parameters from the HEC-2 analysis. The amount of predicted aggradation and degradation is distributed to the verticals of a cross section according to the channel conveyance to yield a set of new cross sections.

The developed mathematical model routes the sediment by size fractions. The transporting capacity of each reach is determined using the average

---

Ramsey M. McDermid was formerly Project Manager and George J. Geiser is Project Engineer with Dames & Moore in Phoenix, Arizona. Daryl B. Simons is President and Principal Engineer and Ruh-Ming Li is Principal Hydraulic Engineer with Simons, Li & Associates in Fort Collins, Colorado.

hydraulic conditions of the reach. The sediment routing procedure is accomplished by applying the sediment continuity equation and considering the size distribution of the upstream sediment supply and the bed material for both surface and subsurface layers.

The model shows the design configuration to be stable during passage of the design hydrograph. The model is also used to predict conditions under which additional flood control works will be required.

INTRODUCTION

As part of the Interstate 10 (I-10) freeway system, a bridge was constructed during 1962 across the Salt River in Phoenix, Arizona. The concrete structure consists of parallel and adjoining twin bridges, each supported by 19 piers based on spread footings. The Arizona Department of Transportation (ADOT) lowered the foundations of four piers on each bridge and installed a 250-ft-wide riprap-lined low-flow channel at the time of construction. The 1,760-ft-long structure spanned the channel and the defined floodplain.

After the I-10 bridge was opened to the public in 1965, its six-lane capacity quickly developed into a vital transportation link connecting central Phoenix and the urban communities to the south and east, as shown in Figure 1. During periods of significant flow in the Salt River most smaller bridges and the numerous unbridged crossings close, thus increasing the importance of keeping the I-10 bridge open for local commuters and emergency vehicles as well as interstate commercial traffic.

During the winter of 1965-66 the first major flow since 1941 damaged the upstream face of the south bridge approach. Successive high flows in 1973, 1978, and 1979 resulted in general riverbed degradation and 250 ft of channel migration to the south, away from the four deeper piers and the low-flow channel. As a result, one shallow pier settled and moved laterally, necessitating closure of the bridge. Dames & Moore was retained by ADOT to provide engineering services to expedite repairs and reopening of the eastbound bridge structure. Repairs consisted of a two-state grout injection program, followed by construction of a new footing and concrete collar to reinforce the pier. The bridge deck was then jacked back to design grade and shimmed.

Foundation support for all of the piers of the Salt River Bridge is provided by spread footings, many of which are based at approximately the same elevation as the pier that failed. Consequently, ADOT was concerned that other pier foundations at the bridge were susceptible to scour damage. Dames & Moore was authorized by ADOT to conduct a phased design study to analyze the susceptibility of the existing pier foundations to scour damage during future floodflows and to develop and compare alternative flood control plans to protect the pier foundations against scour; Dames & Moore then prepared construction plans for the most suitable alternative.

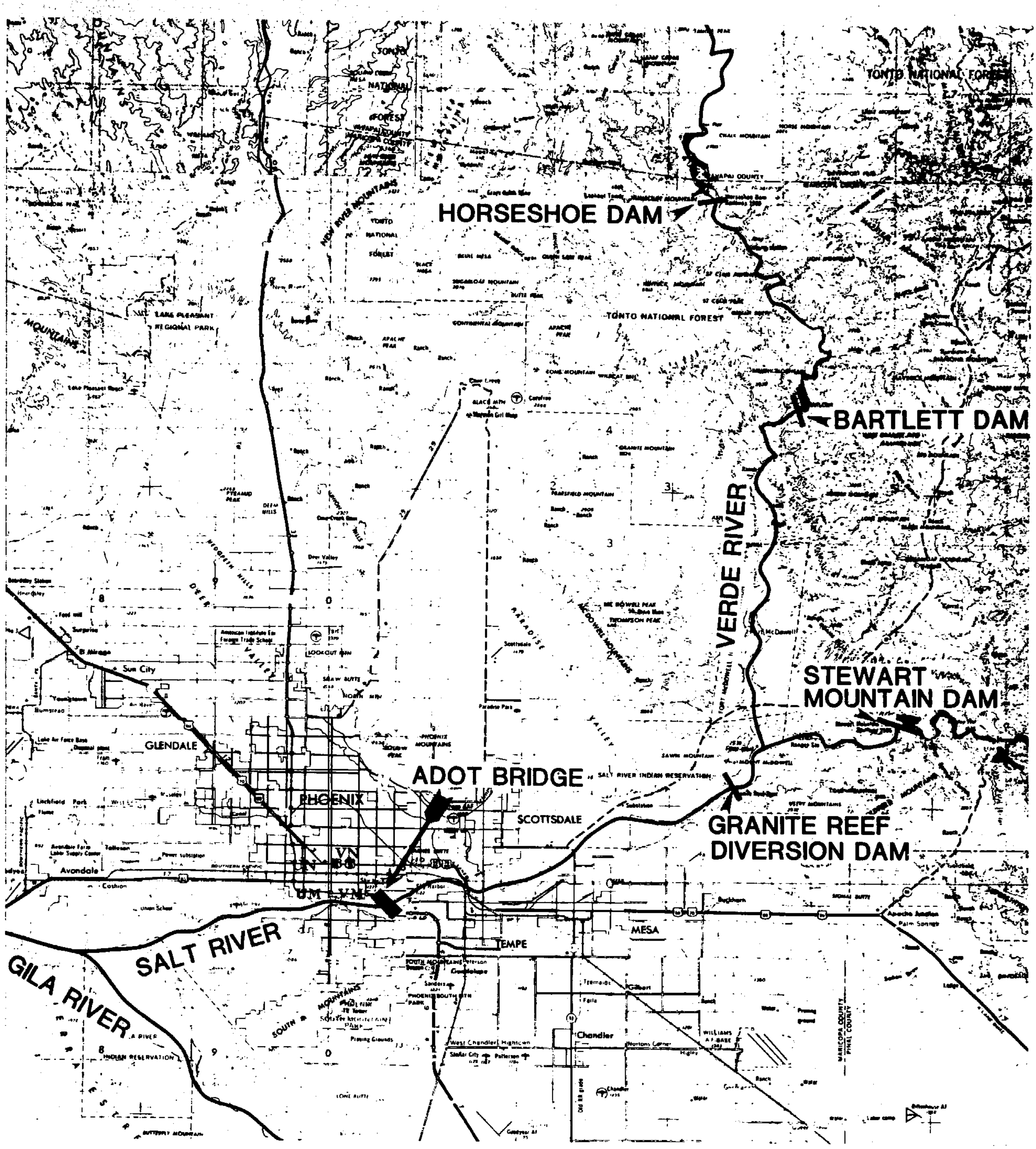


FIGURE 1 Vicinity map of the I-10 bridge.

Dames & Moore contracted with Simons, Li & Associates to provide hydraulic and sediment transport computer modeling of the Salt River. A subsequent progress report included recommendations for interim measures to protect the bridge against scour for a floodflow of up to 130,000 cu ft/s. These recommendations basically suggested reconstructing the bridge's low-flow channel as it was originally constructed in 1962. Reconstruction of the low-flow channel was carried out immediately by ADOT.

Before the design study could be completed, the largest flow of record in the Salt River since the I-10 bridge was built (185,000 cu ft/s) occurred on February 16, 1980. The river's persistent high flow rate caused ADOT to close the bridge on February 19 as a precaution in case the pier footings had experienced damage due to scour. The bridge was closed to traffic for 13 days while investigations were conducted to evaluate the extent of scour adjacent to the bridge pier foundations. Although there was no known structural damage to the bridge, scour holes extending at least to the tops of the pier footings were measured at several locations, and the south bank of the low-flow channel was severely eroded. After the flow rate had decreased, but while the water was still flowing, ADOT placed heavy riprap on the south side of the low-flow channel and around three sets of piers to prevent further erosion of the south bank and to provide some scour protection for the piers in the event that the rate of flow in the river again increased.

On March 10, 1980, ADOT requested that Dames & Moore complete the final design phases on an accelerated basis to enable construction of permanent scour protection measures for the bridge by November 1, 1980.

DESCRIPTION OF THE SALT RIVER

The Salt River drainage area upstream of the I-10 bridge consists of approximately 13,000 square miles of desert and mountainous terrain. Elevations range from 1,100 ft at the bridge to over 11,000 ft in the White Mountains. Vegetation is sparse desert shrubbery in the lower reaches and pine forest in the mountains. A series of six water storage reservoirs has been constructed on the Salt River and its major tributary, the Verde River, since 1905.

Most precipitation occurs as snowfall or general storms in the months of December through February. Summer thunderstorms may produce locally high runoff, but they do not affect the entire drainage area. In the winters of 1977-78 and 1979-80 a series of general storms inundated southern California, Baja California, and southern Arizona. These storms in Arizona filled the reservoirs on the Salt and Verde rivers and set the stage for the flooding that destroyed most of the road crossings in Phoenix.

The Salt River in the desert reach below the mountains follows a wide, braided alluvial streambed. It is typically dry, as runoff from the mountainous headwaters is captured by the reservoirs. The river is relatively steep, with an average gradient through the Phoenix area of approximately 10 ft/mile. The river bed is composed of sand, gravel, and cobbles. Braided streams generally convey large quantities of bed material on steep gradients and are highly susceptible to rapid change.

Man's activities on the Salt River have encroached on the naturally wide alluvial floodplain and on the floodway itself. Encroachments include road fills, transmission tower footings, sanitary landfills, building pads, sanitary sewer crossings, storm sewer outlets, and an airport runway fill. In addition, sand and gravel mining has removed millions of tons of material over the years. The material is typically excavated to the water table; oversize material is rejected and piled in the river or used to form islands around the transmission tower footings.

The water conservation dams on the Salt and Verde rivers have provided incidental flood control storage as they fill, and high flows have seldom been experienced by users of the Phoenix-area riverbed. Because of the natural armoring effect of the cobbles and small boulders, the riverbed has been relatively stable during these low flows. The large volumes of runoff experienced by the watershed in the winters of 1977-78 and subsequent years have filled the reservoirs. The dams have principal spillways designed only for small (less than 4,000-cu ft/s) water supply releases; after a reservoir is filled, floods must pass over the emergency spillway.

FLOOD CONTROL WORKS

The objectives of the final design of the flood control facilities, as defined by ADOT, were to complete designs and to develop construction plans and technical specifications for bridge improvements that would:

1. Provide immediate and permanent protection against riverbed degradation at the bridge crossing.
2. Provide immediate and permanent protection against local scour for the bridge pier footings.
3. Provide a design for the scour protection measures that would allow construction to be completed by November 1, 1980.
4. Satisfy applicable floodplain regulations.
5. Be compatible with drainage and river control facilities that exist or are proposed in the vicinity of the bridge.

The proposed scour protection facilities that were designed to meet these objectives are shown schematically in Figure 2. They consist of three major elements:

1. A series of grade control structures downstream from the bridge to control riverbed degradation at the bridge crossing.
2. Riverbed channelization that will extend upstream from the grade control structure and the bridge. The purpose of the channelization is to control the direction of flow of the water under the bridge and over the grade control structures.
3. Riprap protection over the bottom and side slopes of the portion of the channel passing under the bridge, as well as riprap protection around some of the pier footings situated in the channel. The principal purpose of this riprap protection is to prevent local scour from undermining support of the pier footings.

One of the most critical aspects of the project was the timing of the

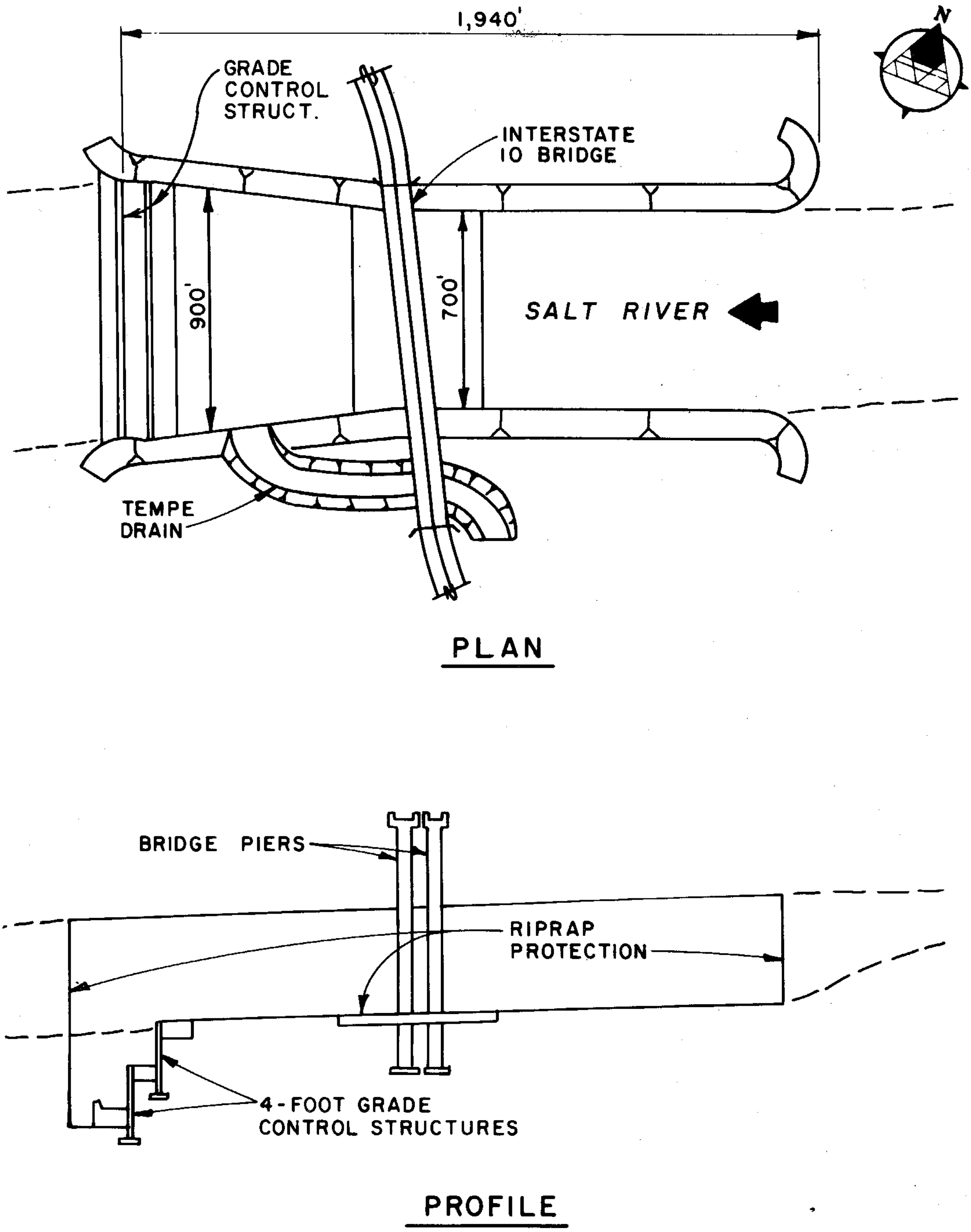


FIGURE 2 Schematic diagram of the Salt River channelization at Interstate 10.

construction of the proposed improvements. Reservoir carryover storage was estimated to be high after the winter of 1979-80, and any significant 1980-81 storm event could cause high runoff and necessitate bridge closure. This situation would be unacceptable to the people and the state of Arizona; flood control improvements had to be in place by November 1, 1980.

HYDRAULIC-SEDIMENT ROUTING ANALYSIS

A hydraulic-sediment routing and scour analysis was conducted to aid in final design of the proposed flood control protection measures. The analysis was conducted by computer using the sediment routing model that was developed during earlier phases of the study. The model consists of a computer program developed by Simons, Li & Associates for routing sediment by size fraction. It is used in conjunction with the HEC-2 Water Surface Profile program developed by the Corps of Engineers. The model routes sediment by size fractions for discrete time steps of the hydrograph. The channel bed's response to flow conditions is available at all times during the passage of the hydrograph.

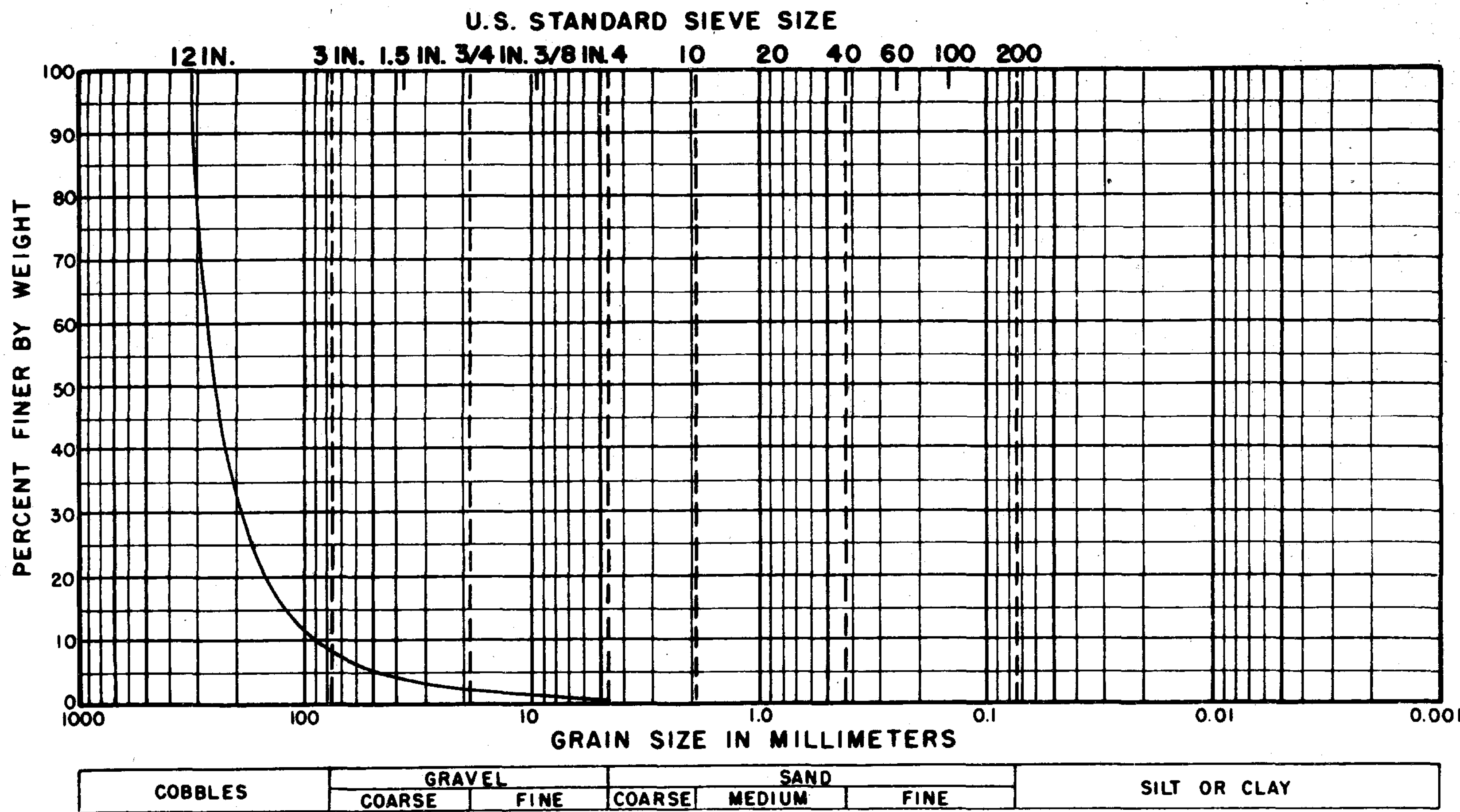
The reach of river modeled for final design extended from approximately 15,000 ft downstream to 28,500 ft upstream from the I-10 bridge. The most recent topographic mapping and river cross section data were provided by ADOT and were used to provide input data for the model. The river cross sections used in the computer analysis include sections 1 through 22, which are shown in Figure 3. Each reach is composed of similar cross sections, slopes, and sediment characteristics.

Both surface and subsurface samples of the bed material were collected and analyzed by Dames & Moore in 1979. The composite size distributions for both surface and subsurface samples are given in Figures 4 and 5. They did not change significantly during the 1980 flood. The surface sample shows a significant armor layer. Field observations verify that it is difficult to collect representative samples of bed material in the Salt River. For simplicity, the following characteristics of the bed material are adopted for the scour analysis. The surface layer has a D_{50} (median diameter) of 237 mm and a σ (gradation coefficient) of 1.6. The subsurface layer has a D_{50} of 123 mm and a σ of 7.0.

The response of the river during the design flood was evaluated for two basic conditions: existing conditions, and future conditions assuming that the first two grade control structures are built and the river is channelized at the bridge as proposed. The existing conditions were evaluated to provide a base for comparing the response of the river to the proposed scour protection facilities. The future conditions were evaluated to aid in the layout and sizing of the proposed scour protection measures and to investigate their stability during the design flood. The water surface elevation predicted for the design flow for future conditions was compared with that estimated for existing conditions to determine compliance with federal floodplain regulations.

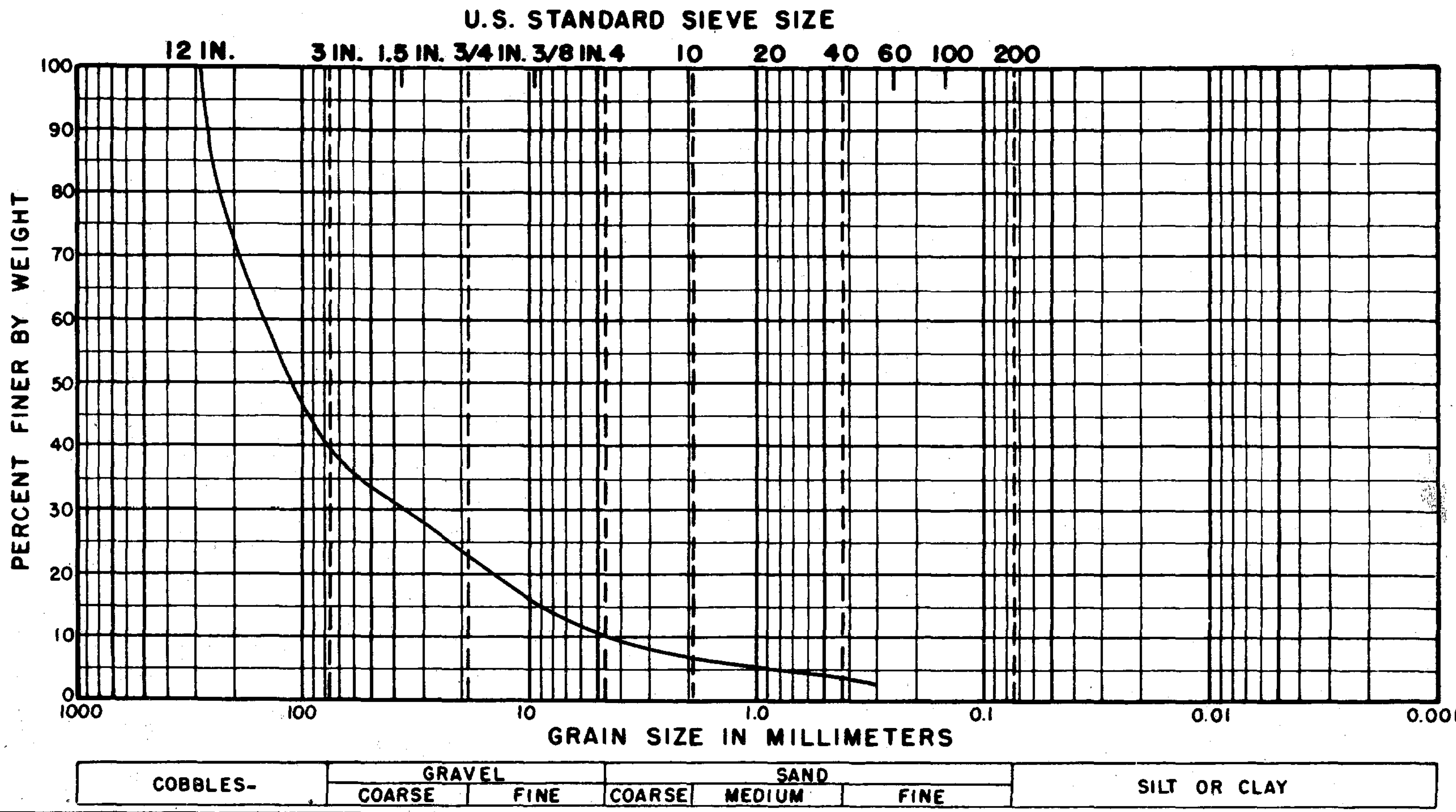
River Distance	Cross Section Number	Location	Reach Definition
28,480	22	Upstream boundary	
25,880	21		8
24,195	20		
23,545	19		7
21,580	18		
20,110	17		6
18,060	16		
17,030	15		
16,000	14		5
15,490	13		
14,990	12		
14,890	11	I-10 bridge	4
14,790	10		
14,360	9		
13,945	8		3
12,845	7		
11,145	6		
9,445	5		2
7,380	4		
5,280	3		
2,600	2		1
0	1	Downstream boundary (Seventh St.)	

FIGURE 3 Index map for the Salt River in the vicinity of the I-10 bridge ("as-is" cross section).



	DEPTH	CLASSIFICATION		NAT. WC	LL	PL	PI	
	SURFACE							

FIGURE 4 Gradation curve for composite surface sample.



	DEPTH	CLASSIFICATION		NAT. WC	LL	PL	PI	
	12 FEET							

FIGURE 5 Gradation curve for composite subsurface sample.

SALT RIVER/I-10
8/79

Existing Conditions

Results of the analysis for existing conditions indicated that significant degradation of the riverbed at the bridge crossing would have been expected due to the design flood. The time-lapse change in the general scour depth at the I-10 bridge during the design flood is shown graphically in Figure 6. The total predicted depth of general scour was slightly more than 11 ft.

In addition to general scour, some local scour would have been expected at the bridge due to disturbances in the water flow generated by the piers. Assuming that the flow of the river was practically parallel to the stems of the piers, the depth of local scour was estimated to be in the range of 5 to 7 ft. If a 15-degree angle of attack was assumed for the flow approaching the piers, the local scour would have been expected to increase to 13 to 18 ft.

Generally, local scour is added to general scour to estimate the total depth of potential scour. Therefore the total potential scour depth at the bridge crossing was estimated to be between 16 and 29 ft, depending on the angle of attack of the water on the piers.

The water surface elevation versus flow rate at the bridge for existing conditions is presented in Figure 7. The looping effect shown in the figure is due to movement of sediment. The calculated water surface elevation for the design flow of 176,000 cu ft/s is approximately 1,103 at the bridge crossing. For comparison, field measurements taken at the bridge during the February 1980 flood indicated an approximate maximum scour depth of 16 ft. The water surface was measured at an elevation of approximately 1,103.7 with an estimated flow rate of 170,000 cu ft/s. This appears to confirm that the results of the computer model analysis are sufficiently precise.

Channelized Conditions

The computer model was also used to evaluate the response of the river assuming that the first two grade control structures were installed and the riverbed channelized for a distance of 875 ft upstream from the bridge. The analysis indicated that the depth of general scour at the bridge crossing due to passage of the design flood would be insignificant. In fact a slight amount of aggradation would probably occur. The depth of local scour around the pier was estimated to be in the range of 4 to 6 ft. This assumes that the proposed channelization would control the direction of flow of the river and that the angle of the flow's attack on the piers would be negligible. To provide additional protection against undermining of the pier footings by local scour, a riprap blanket over the channel bottom was recommended. The leading edge of the guide banks of the bridge channelization would be expected to experience approximately 7 ft of local scour. Therefore slope protection was required for the channel side slopes to guard against the local scour.

The water surface elevation versus flow rate at the bridge crossing for future conditions is shown in Figure 8. Because of the relatively stable bed configuration, no looping effect due to movement of sediment is shown in Figure 8. The water surface at the peak flow rate of 176,000 cu ft/s should

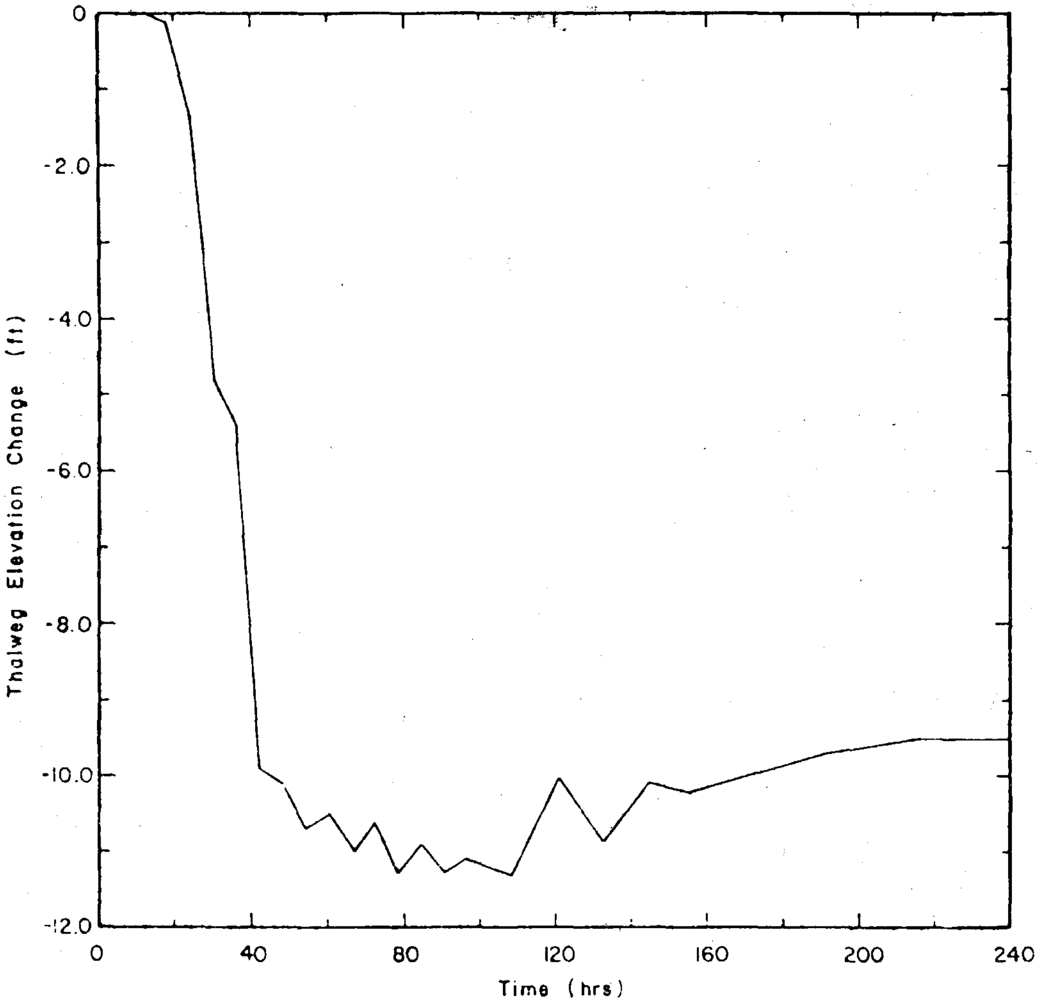


FIGURE 6 Degradation-aggradation at bridge during the design flood.

be at an elevation of approximately 1,101, and the average channel velocity should be about 11.5 ft/s.

The water surface elevation at peak flow for future conditions is approximately 2 ft less than that calculated for existing conditions. Consequently, the proposed protection measures should comply with floodplain regulations.

ALTERNATIVE CONSTRUCTION MATERIALS

Drop Structures

Several alternative construction materials and methods of construction were considered for the drop structures. These included:

1. Soil cement base
2. Riprap base
3. Gabion base

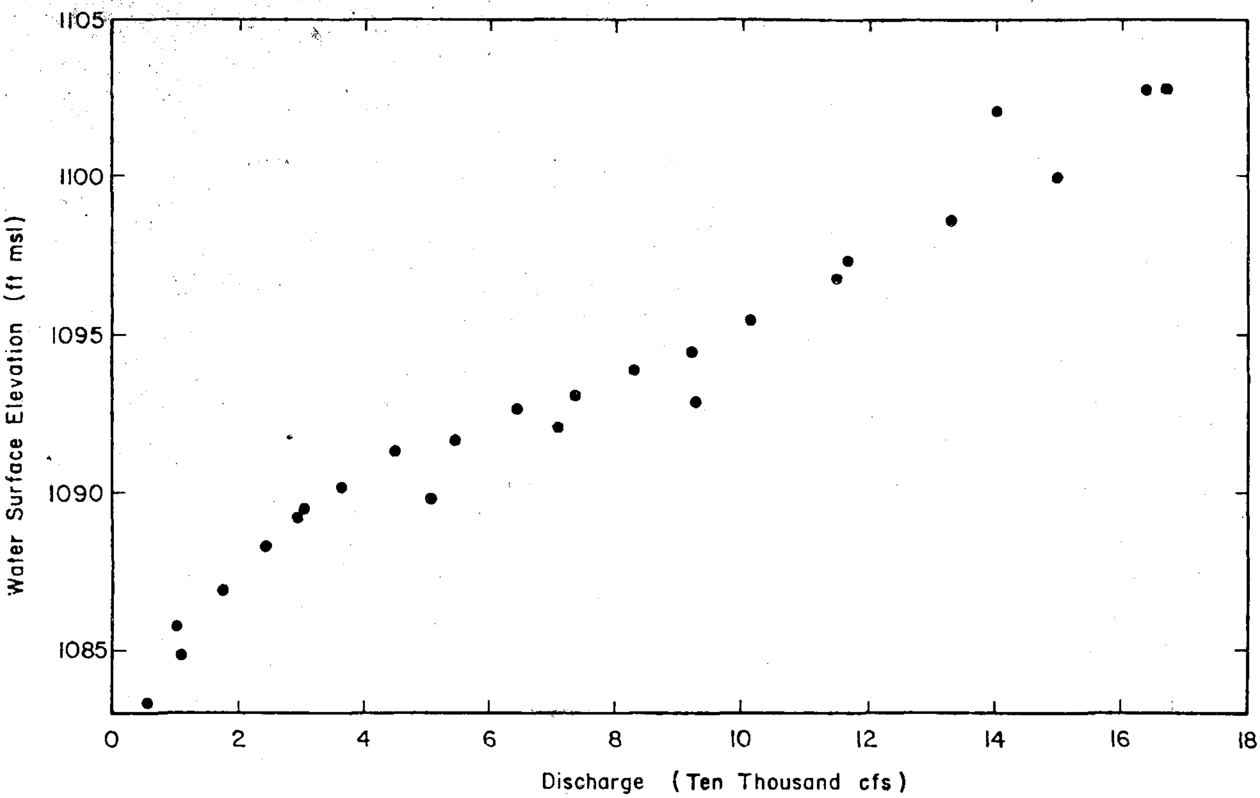


FIGURE 7 Stage-discharge relationship at bridge (as-is condition).

4. Steel bin retaining wall base
5. Reinforced concrete retaining wall
6. Grouted river rock base

Designs of sufficient detail to allow comparison of the alternative types of construction were prepared, and the cost for each alternative was estimated.

Based on this cost comparison, a reinforced concrete retaining wall and gabion base drop structure were considered the most economically attractive. In our opinion the abrasion and corrosion resistance offered by the wire gabion baskets would not be as suitable as concrete for the conditions and environment of the Salt River. Therefore the reinforced concrete retaining wall and grouted riprap apron were selected for the grade control structures.

Channel Side Slope Protection

The calculated velocity of flow in the proposed channel in the vicinity of the bridge is 11.5 ft/s for the design flow rate of 176,000 cu ft/s. This velocity is an average value for the channel; it does not consider higher velocities caused by local concentrations of flows.

A velocity of 15 ft/s was assumed for design of the side slope protection. Armoring of the side slopes of the channel was required to provide protection against erosion. The armoring was also extended to protect

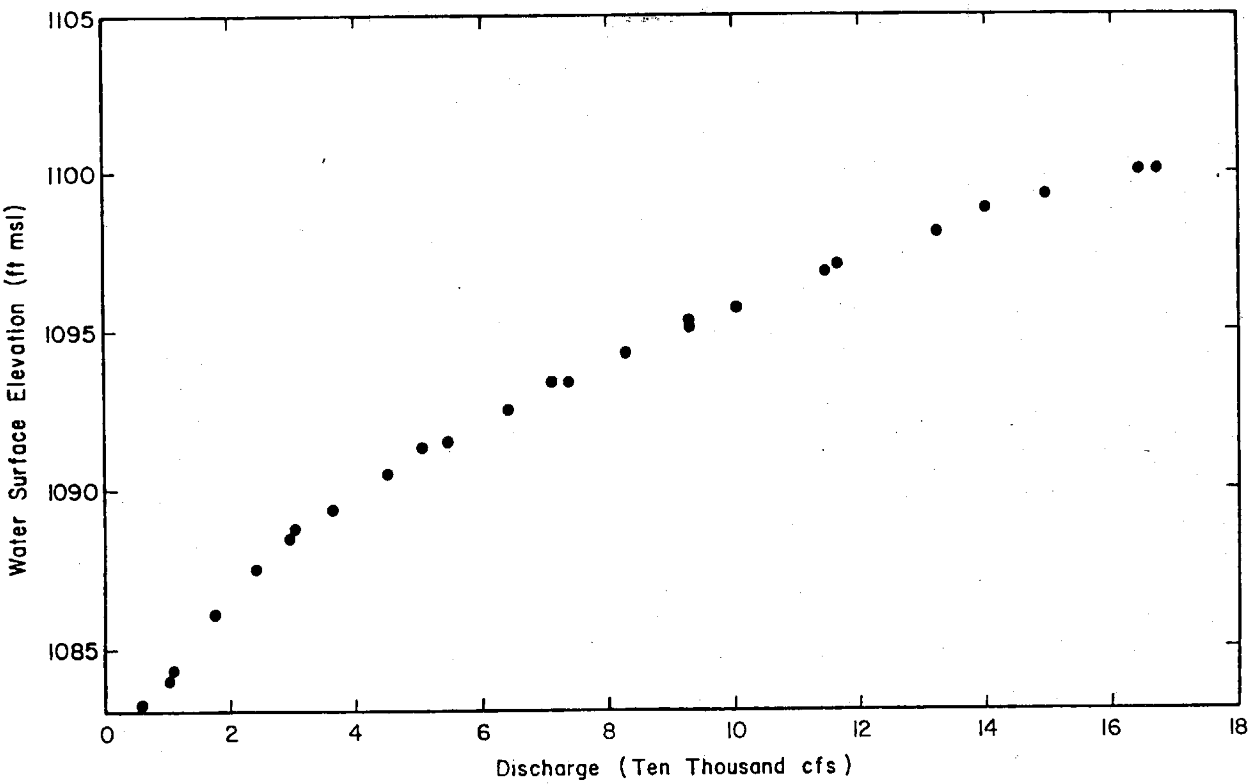


FIGURE 8 Stage-discharge relationship at bridge for future conditions.

the toes and tops of the slopes to prevent undercutting and topcutting, respectively. Alternative construction materials were also evaluated for channel side slope protection. These were:

1. Gabion baskets
2. Riprap
3. Fabriform mats (grout-filled nylon forms)
4. Grouted riprap
5. Soil cement

A comparison of the estimated unit cost of each indicated that soil cement, fabriform mats, and riprap were the most economically attractive alternatives.

There was some doubt regarding the durability of the fabriform mat slope protection under the heavy abrasive action encountered during flows of the rate and velocity that may be encountered in the Salt River. Failure of only one mat under conditions of high flow could lead to rapid and complete slope failure of the channel bank. Because of this concern the fabriform mat alternative was dropped from further consideration.

Of the remaining two alternatives riprap was preferable to soil cement, in our opinion, because of its excellent durability. Riprap also has the ability to settle and redistribute its weight without detrimental effects on its performance as slope protection. However, it is not available locally in the Phoenix area and thus is expensive.

Discussions with the Portland Cement Association revealed that soil cement has been used successfully for side slope protection for similar purposes and its performance has been satisfactory. Therefore a properly designed and constructed soil cement slope lining should adequately protect against erosion by occasional floodflows in the river. The cost of the soil cement slope lining can be reliably estimated, in contrast to the case with riprap, since the materials for its construction are readily available.

Based on the above considerations, the riprap and soil cement alternatives both had important advantages. Both were considered suitable from the technical standpoint, but the riprap was preferable. To allow for the possibility that a contractor could provide riprap at a price competitive with soil cement, alternative bids were solicited. The low bidder chose riprap from a nearby open pit mine.

ADDITIONAL STAGES OF DROP STRUCTURE CONSTRUCTION

The first two drop structures constructed are designed to protect in the immediate and near future against general riverbed degradation. As discussed earlier, future sand and gravel mining activities may require additional structures to be constructed downstream. The following sections discuss ways to determine when additional drop structures should be constructed.

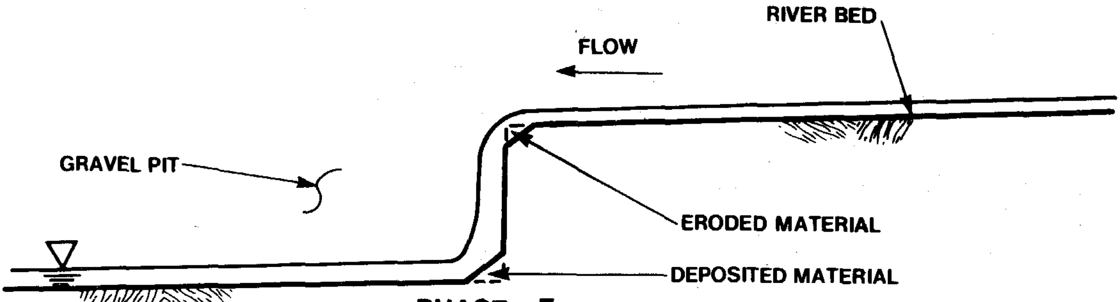
Failure Mechanism

The governing processes of erosion and deposition due to a large downstream pit in an alluvial river are illustrated in Figure 9. Headcutting of future sand and gravel pits downstream is the most likely element of the erosion-deposition process that could damage the initial two drop structures. Additional drop structures may be required in the future to guard against this.

As shown in Figure 9, headcutting will develop in three phases. The first phase consists of the actual excavation of the pit in the riverbed, which sets the stage for headcutting when water begins to flow in the river. When the water flows at a high velocity over the edge of the pit, it rapidly erodes the edge and begins headcutting upstream. The eroded material is deposited in the pit. This phase may occur at the start of high flows, or it may occur during continuous low flows.

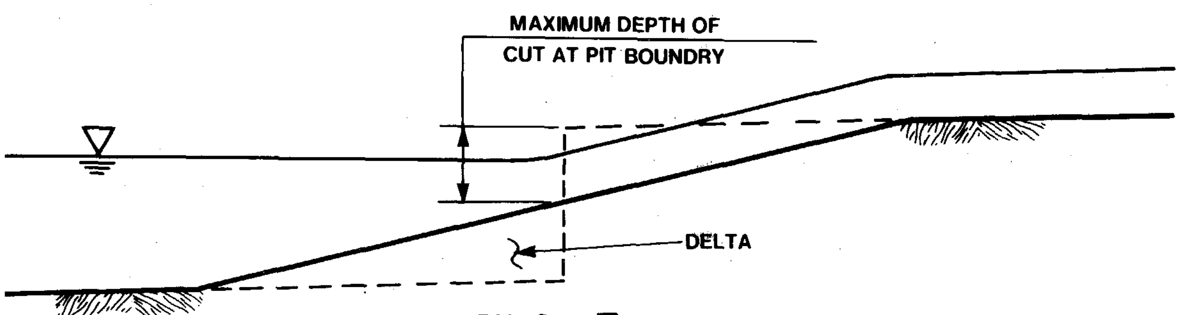
During the second phase the material eroded from the edge of the pit and upstream has built up a delta in the pit. The partially filled pit also creates a reservoir, which causes the sediment load normally carried by the stream also to be deposited in the delta. The head of the delta will reach the crest of the headcut; thus the maximum depth of headcut at the pit boundary will be attained. Erosion and deposition may continue due to the locally steep energy grade line.

The third phase of the erosion-deposition process consists of continued



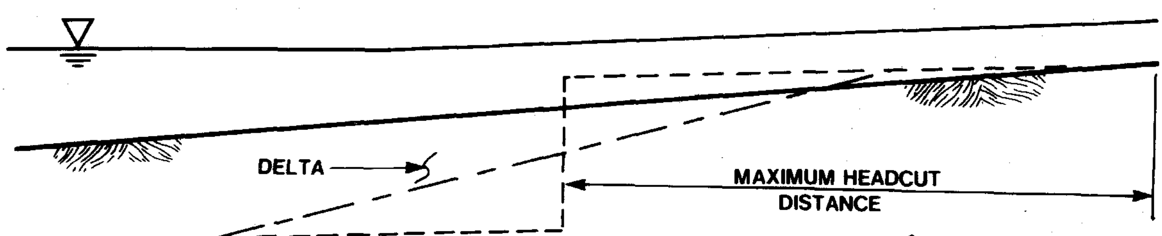
PHASE I
START OF HEADCUT

Low flows or initial portions of larger flows rapidly erode the edge of the pit and deposit the material at the toe of the slope.



PHASE II
MAXIMUM HEADCUT AT EDGE OF PIT

Continued low flows or larger flows have deposited material in the pit up to the depth of the cut at the edge of the pit. The river slope immediately upstream of the pit is steep and erosion and deposition will continue.



PHASE III
MAXIMUM HEADCUT DISTANCE

(NO SCALE)

The river and gravel pit have reached an equilibrium condition for the particular sequence of flows being considered. Continued flows of water and sediment will cause the bed to remain stable or aggrade.

FIGURE 9 Schematic diagram of erosion and deposition at gravel pit.

headcut erosion upstream and continued deposition in the pit. At some maximum headcut distance the bed slope is reduced or backwater effects from the reservoir cause the water-sediment system to reach an equilibrium condition. The riverbed will remain relatively stable during continued flow at the same rate or will aggrade during lesser flows.

"Danger" Criteria

If the propagation of headcutting upstream from the pits reaches the downstream drop structure, and if the depth of headcutting at the downstream drop structure reaches its foundation level, the foundation support for the drop structure could be undermined. The riprap apron that is planned to extend downstream from the drop structure will partially protect against undermining of the foundation of the drop structure by headcutting. However, the riprap apron is not intended to provide permanent protection for the drop structure. Instead, additional drop structures must be constructed downstream as the location and geometries of pit excavation downstream dictate.

To aid ADOT in determining when additional drop structures will have to be constructed, several hypothetical pit locations and geometries were evaluated during the hydraulic and sediment modeling analysis. It was assumed in the analysis that 6 ft is the maximum headcutting depth allowable at the drop structures. This depth is measured from the top of the upstream drop structure to the riverbed level immediately downstream from the second drop structure, as illustrated in Figure 10. The hypothetical pit locations and depths required to cause headcutting of 6 ft at the drop structures under design floodflow conditions were then predicted based on the modeling analysis. The results of this analysis are summarized in Table 1 and Figure 10.

Table 1 gives the maximum headcut distance, the bedslope at maximum headcut distance, and the distance at which the headcut depth is 6 ft for several pit geometries. The distances and slopes are shown graphically in relation to the drop structures in Figure 10. Bed slopes at which equilibrium conditions are reached are steeper than the natural river slope due to the effect of backwater from the pit. A pit size of 60 acres was assumed for each of the hypothetical pits. Different pit sizes and flood hydrographs would result in different predicted headcut depths at the drop structures.

Presented in Figure 11 is a graphical summary of those pit depths and distances from the upstream edge of the pits to the downstream drop structure that are predicted to cause different depths of headcutting at the drop structure during the design flood. ADOT can use this graph to assess the potential impact that downstream mining activities may have on the safety of the drop structures and to determine when construction of additional grade control structures may be needed.

It should be emphasized that Figure 11 is based on assumed pit depths and distances and the existing topography of the riverbed. Additionally, the headcut distances shown on the plate assume passage of the design flood of

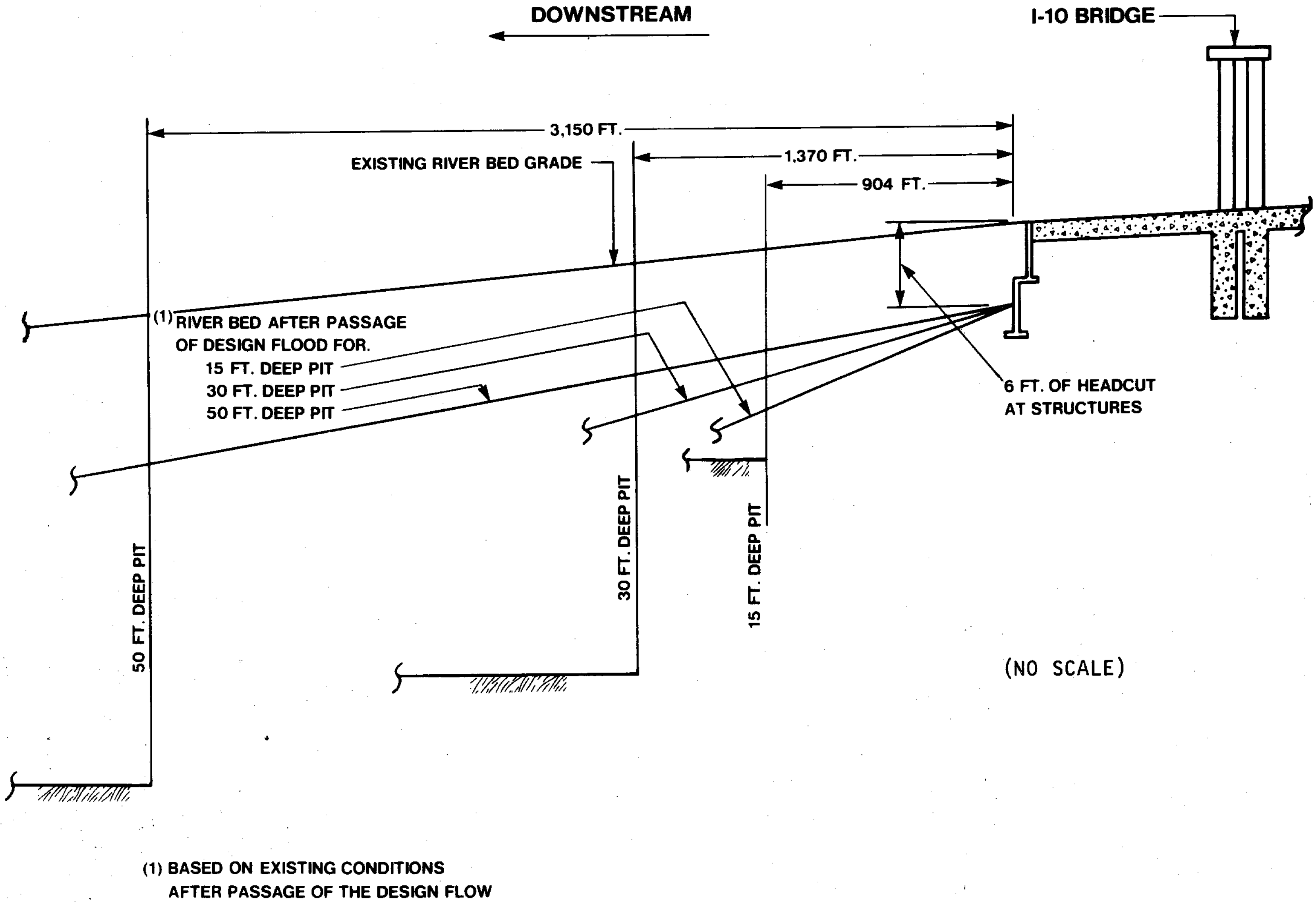


FIGURE 10 Alternativé pit geometries for a 6-ft headcut.

TABLE 1 Maximum Allowable Pit Depths[a] and Distances from Drop Structures

Pit Depth (ft)	Pit Volume (acre-ft)	Maximum Headcut Distance[b] (ft)	Bedslope at Maximum Headcut Distance[c] (ft/ft)	Distance at Which Headcut Depth Is 6 Ft (ft)
15	900	1,400	0.0121	904
30	1,800	1,940	0.0105	1,370
50	3,000	3,750	0.0100	3,150

[a]Based on computer model results.

[b]Distance includes 1.5 factor of safety.

[c]Includes natural river slope.

176,000 cu ft/s and 11 days duration. A series of smaller flows and downstream pit excavation prior to passage of the design flood may erode material between the lower drop structure and pits, thereby reducing the available material needed to create the delta for a higher flow condition. If this occurs, additional modeling analysis should be performed to reevaluate the relationship between the safety of the proposed drop structures and future mining operations.

It was recommended that ADOT make at least a yearly topographic survey of conditions between the I-10 bridge and Central Avenue. In particular, a survey should be conducted after flows of any significant magnitude have passed. Based on results of the topographic survey, the following analyses should then be conducted.

1. Determine the thalweg profile and compare it with the previous profile.
2. Determine the pit volumes and their effective depths and distances from the downstream grade control structure.
3. Plot the effective depth of each pit and its distance from the downstream grade control structure on the graph presented in Figure 11.

If the thalweg profile shows a significant loss of material downstream from the drop structures, or if the depths and distances of gravel pits plot in or above the crosshatched area shown in Figure 11, it is recommended that a hydraulic-sediment routing model analysis be conducted to reevaluate the potential impact that the sand and gravel pits may have on the safety of the proposed drop structures. Based on this reevaluation, a decision can be made on whether or not to construct additional drop structures. When and if additional drop structures are required, it is recommended that a physical

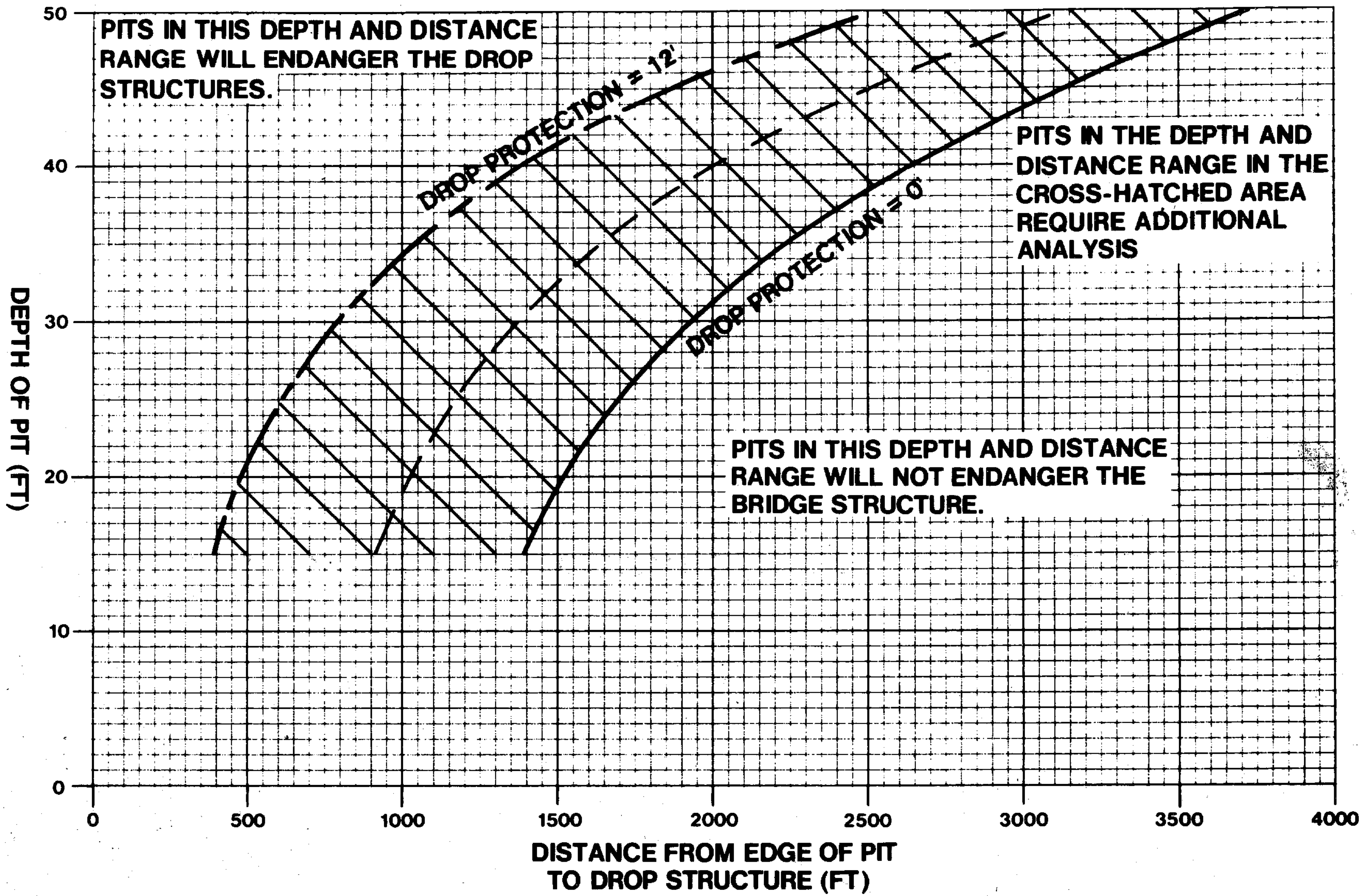


FIGURE 11 Evaluation graph of drop structure safety. (Based on assumed pit sizes of 60 acres. A series of smaller storms will require reanalysis.)

model study be carried out to aid in their design. A series of low-drop structures, such as have been designed for the initial stage of construction, are not amenable to rigorous hydraulic analysis. A physical model is the best means of confidently predicting their behavior under various flow conditions.

COASTAL WINTER STORM DAMAGE,
MALIBU, LOS ANGELES COUNTY, WINTER 1977-78

by George A. Armstrong

In the winter of 1977-78 a combination of high waves, local storm surges, and high tides caused $18 million in damages along the California coastline. The southerly and southwesterly facing beaches in the Malibu area were especially hard hit by waves passing through the open wave windows between offshore islands. These waves broke against beaches, seawalls, and other structures, causing damages of between $2.8 and $4.75 million to private property alone.

The amount of erosion resulting from a storm depends on the overall climatic conditions and varies widely from storm to storm. Protection from this erosion depends largely on the funds available to construct various protective structures that can withstand high-energy waves. To prepare adequately for severe storms, property owners should combine their efforts to upgrade seawalls and other protective devices. Such cooperation is not yet common.

The dissemination of a publication describing coastal processes and protection techniques could inform the public of the hazards of living in coastal areas and help them select methods of shore protection.

INTRODUCTION

During the winter storms of December 1977-April 1978 California suffered significant coastal damage. During this period a series of severe storms battered the California coast. The destructiveness of these storms came from a combination of high astronomical tides, strong onshore winds, high storm waves, and excessive precipitation. This combination of environmental factors caused extreme erosional conditions along the entire California coast. This type of storm damage is not new to the southern California coast, and historical meteorological data suggest that wave damage will continue to occur along the coast at infrequent periods in the future. The resulting $18 million of damages from high sea swells and high tides exceeded previous

---

George A. Armstrong is with the Beach Erosion Branch of the Department of Boating and Waterways for the Resources Agency in Sacramento, California.

losses during the last few decades. The damages statewide are tabulated in Tables 1 and 2. This paper describes the causes and consequences of the dynamic conditions that led to the storm damage along the California coastline, with a special assessment of conditions, as reported by the U.S. Army Corps of Engineers and the California Coastal Commission, for the portion of the Los Angeles County shoreline near Malibu.

CONDITIONS OF THE 1977-78 WINTER

The natural conditions that damaged coastal structures along the Los Angeles County shoreline in the vicinity of Malibu consisted of a combination of high waves, local storm surges, and high tides. During the 1977-78 winter storm period a large high-pressure system over Alaska and western Canada caused storm centers to be moved 1,000 to 1,500 miles due west of central California, further south than their normal locations. Under these conditions the more southerly high directed a series of storms to hit California. Strong westerly and southwesterly winds generated large swells. The swells passed through the open wave windows between the offshore islands (Figure 1), reaching the Malibu area with an unusually large amount of energy. Table 3 gives the wind and wave data for some of the storms. Due to the location of the storm centers, the southwesterly facing beaches from Point Dume to Santa Monica experienced extremely high waves. Table 4 gives the heights of breakers along the southern California coast.

The strong winds associated with these frequent storms caused superelevations of sea level (storm surges) that approached 2 ft at some locations. The combination of storm surges and extremely high tides allowed high waves to pass over offshore bars without breaking. The highest predicted tides (Table 5) were on January 7-10 in the Los Angeles area. Thus the storm surges, combined with high tides and high sea swells, created a condition that allowed large waves to break on the beach, against seawalls, and on other structures during this period.

The most important characteristic of the 1977-78 winter storms was their persistence. Virtually every storm generated in the North Pacific hit the California coast. Beaches along the Malibu coast are usually 50 to 150 ft wide and provide a good protective beach for one or two storm events during a normal winter season. The usual horizontal loss of beach face during a single storm is about 50 ft. Between December 1977 and March 1978 the repeated wave attacks overtopped existing beach berms and subsequently eroded and lowered the beaches by 10 to 15 ft in elevation. This extreme loss of protective beach made all coastal structures vulnerable to the high-energy waves breaking at the base of or directly on the structures. These repetitive high-energy waves caused the older seawalls along Malibu Beach to fail.

MALIBU AREA DAMAGE

Private Sector

During the winter of 1977-78 high tides and large storm waves in the Malibu area caused damage of between \$2.8 and \$4.75 million to private

TABLE 1 Summary of Wave Damage Costs by Region, Winter 1977-78 (dollars)

Region	Private Damage	Public Assistance to Private Parties[a]	Public Damage	FDAA Aid[b] for Public Property Damage
North Coast	85,000		4,200,000	3,261,000[c]
North Central Coast	604,000	66,000	510,000	
Central Coast	853,200	--[d]	196,000	137,000
South Central Coast	500,000	8,000	1,742,800	8,000[e]
South Coast	2,150,000[f]	96,430	606,100	302,300
San Diego Coast	700,000[g]	4,500	1,026,300	273,700[h]
Total	4,852,200	174,930	8,280,800	3,982,000

Note: Costs are estimates of losses, repairs, and/or emergency work.

[a] Does not include Small Business Administration loans.
[b] The Federal Disaster Assistance Administration (FDAA) approved grants through July 25, 1978.
[c] Includes $3,225,000 in Federal Highway Administration aid.
[d] Public assistance was given, but the cost is not yet known.
[e] FDAA grant given to the City of Santa Barbara to remove destroyed homes.
[f] Includes $800,000 that may have been a combination of wave and other damage.
[g] San Diego County private property damage not well documented.
[h] Does not include an application from the City of Oceanside for $3.5 million from the FDAA.

Source: U.S. Army Corps of Engineers, San Francisco District.

TABLE 2 Summary of Wave Damage Costs by County, Winter 1977-78 (dollars)

Coastal County	Private Damages	Public Aid Involved	Public Damages	Federal Aid
Del Norte	Wave damages reported; cost was not determined			
Humboldt			3,500,000	3,500,000
Mendocino	85,000		552,000	552,000 36,500[a]
Sonoma	No damage reported			
Marin	242,000	138,400[b] 66,000[c]	10,000	
San Francisco	No damage reported			
San Mateo	25,000		145,000	
Santa Cruz	844,500	482,100[b] 8,741[f]	209,900	
Monterey	8,000		137,000	137,000
San Luis Obispo	335,000		335,000	--[e]
Santa Barbara	260,000		36,900	36,900
Ventura	300,000	1,141[c]	1,500,000	135,000
Los Angeles	3,000,000	2,824,700[b] 89,300[c] 70,000[d]	624,900	269,300
Orange	130,000	92,900[b]	108,500	101,200
San Diego	922,000	418,100[b]	4,000,000	516,600
Total	6,151,500	4,191,382	11,865,500	4,784,500

Total of reported damages = $12,009,000

Total of reported financial assistance (Small Business Administration loans, federal grants, local government aid) = $8,974,441

Subsidy (from SBA loans, federal grants, and local governments) = $2,825,441

[a] State aid.
[b] SBA loans.
[c] Local government expenditure.
[d] National Guard.
[e] Aid given but amount not known.
[f] U.S. Army assistance.

Source: California Coastal Commission.

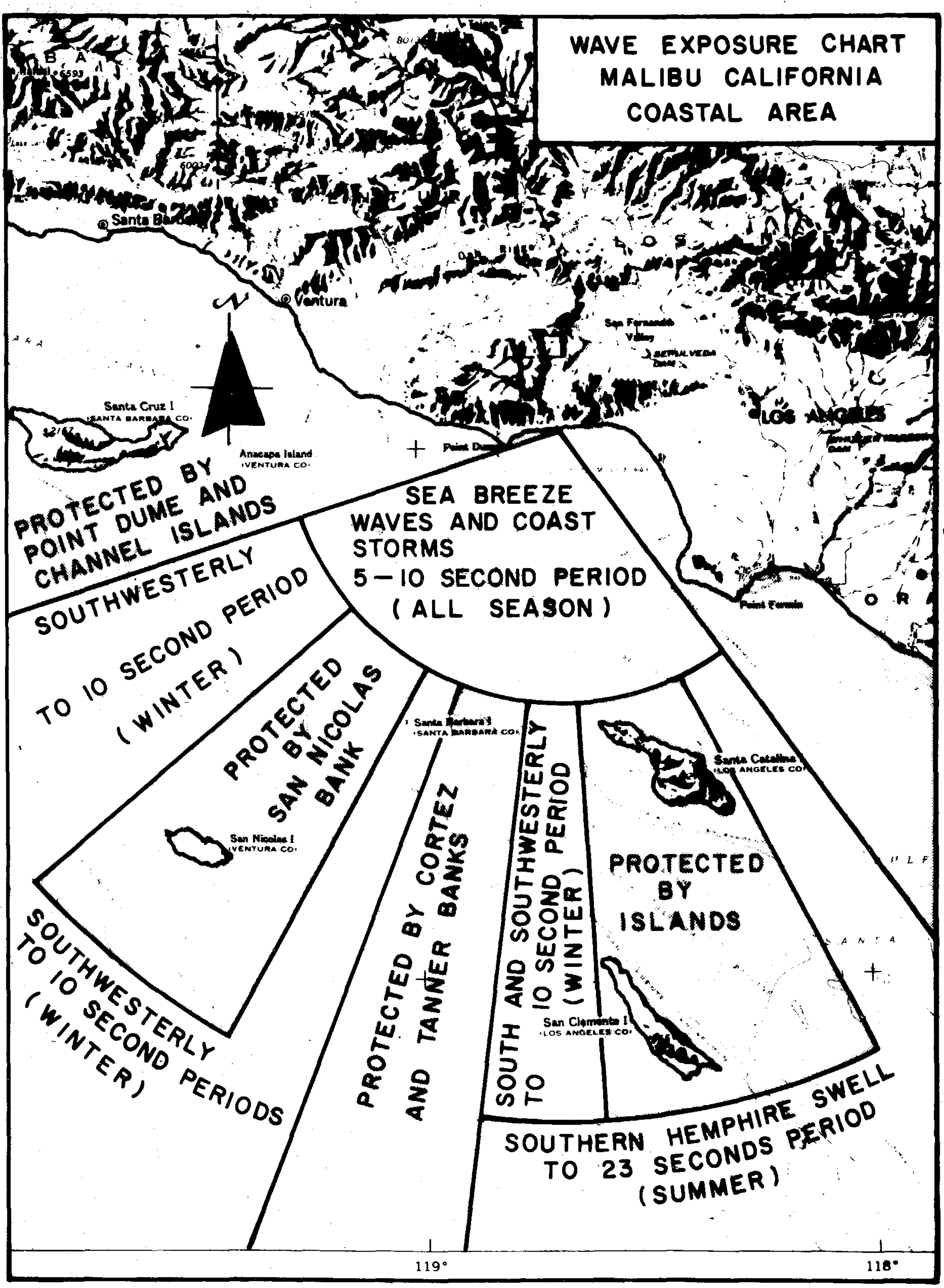


FIGURE 1 Wave exposure chart for the Malibu coastal area.

TABLE 3 Wind and Storm Wave Data, January and February 1978

	Wind		Significant Wave		
Date	Speed (knots)	Direction	Height (ft)	Period (sec)	Direction
Jan. 9	35	SW	14	18	SSW
Jan. 13	35	SW	21	14	S
Jan. 16	50	SW	18	16	WSW
Feb. 9	45	S	20	16	SSW
Feb. 10	40	WNW	16	12	SSW
Feb. 13	40	NW	17	16	SSW

Note: Wind data measured at Farallon Islands (NOAA). Wave data obtained 60 miles west of Golden Gate.

Source: U.S. Army Corps of Engineers, San Francisco District.

TABLE 4 Maximum Breaker Heights, December 12, 1977-March 12, 1978

Maximum Observed Height (equal to or greater than) (ft)	Duration (days)
15	6
13	7
12	15
11	16
10	23
9	25
8	31
7	53

Source: U.S. Army Corps of Engineers, San Francisco District.

TABLE 5 Predicted Tide Heights at Los Angeles (Outer Harbor), Winter 1977-78

Month	Day	Time (hour minute)	Height (ft)
January	6	0622	6.4
	7	0710	6.8
	8	0755	7.0
	9	0841	7.0
	10	0927	6.8
	11	1015	6.2
February	4	0611	6.2
	5	0701	6.5
	6	0747	6.7
	7	0835	6.7
	8	0918	6.4
March	6	0656	6.0
	7	0741	6.1
	8	0827	6.0

Note: Heights 6 ft or higher are shown. Storm surge was estimated at 2.0 ft. The maximum observed tide at Golden Gate was 8.26 ft (mean lower level water datum); the predicted tide was 6.8 ft.

Source: U.S. Army Corps of Engineers, San Francisco District.

property. The National Guard and the Los Angeles County Fire Department, Sheriff, Engineer, and other departments provided $159,000 in emergency assistance to homeowners and businesses. Small Business Administration (SBA) loans approved for the repair of wave-related damage at Malibu totaled about $2.8 million. The following description of damage at Malibu is based on personal observations, correspondence with government agencies and private consultants associated with the Malibu area, and data supplied by these sources to the California Coastal Commission.

The Malibu Film Colony, one of the hardest hit coastal areas in the state, received the most publicity. This was due to the number of movie stars and other celebrities owning homes in the area. The coastline faces south to southwest and was directly exposed to the large storm waves. Many of the beach homes are built right on the beach and are 30 to 40 years old. Protective works, such as bulkheads, proved ineffective or had deteriorated to the point that any storm waves could cause severe damage. Losses include severely damaged or destroyed bulkheads, seawalls, beach access stairs, teahouses, decks, patios, piers, and pilings, numerous broken windows, and considerable interior flooding.

Similar damage occurred at Seacliff and Pothelly beaches (in Santa Cruz County), Favia Beach (in Ventura County), and along Oceanside and Del Mar beaches (in San Diego County). More than 140 residences along the Malibu coastline were damaged by high waves, tides, and storm surges.

The photographs of Figures 3-9, indexed to the mileage of the map shown in Figure 2, illustrate some of the damage at Malibu.

SBA records indicate that 138 property owners on the seaward side of Pacific Coast Highway, Malibu Road, and Malibu Colony Drive filed applications for $4.3 million in disaster loans. Other beach areas at Malibu had applications for $450,000 in damage. Public expenditures used to protect private property included those incurred by the National Guard and the Los Angeles County Fire, Sheriff, Road and Building, and Safety departments. These agencies provided personnel and equipment to advise property owners and construct temporary protection devices. Student volunteers from nearby Pepperdine University also helped Malibu residents install sandbags and make emergency repairs.

The National Guard provided about 1,871 man-days from March 4 through March 7 in protecting homes and assisting residents. The National Guard operations cost approximately $70,000 and came from the state emergency fund.

The Los Angeles County Sheriff's Department from January through March also aided Malibu residents during each winter storm. The Fire Department does not record man-hours and costs of normal emergency activities except when it plans to apply for reimbursements from other agencies, but cost estimates supplied to the Santa Monica Mountains Comprehensive Planning Commission indicate that the Fire Department's total cost of involvement exceeded $70,000 at Malibu.

The Los Angeles County Road Department purchased about 20,000 to 30,000 sandbags at a cost to the county of approximately $3,400. The sandbags were used by county agencies, the National Guard, residents, and volunteers to protect homes along the beach.

Public Sector

A storm drain ocean outfall was severely damaged on February 9, 1978. The structure is maintained by the Los Angeles County Flood Control District. Large storm-driven waves contributed to the failure of the outlet headwall and 30 ft of 60-in.-diameter reinforced concrete pipe. The district's cost for labor, equipment, materials, and associated emergency work completed immediately after the failure was $4,200. The cost of repair and replacement will exceed $100,000.

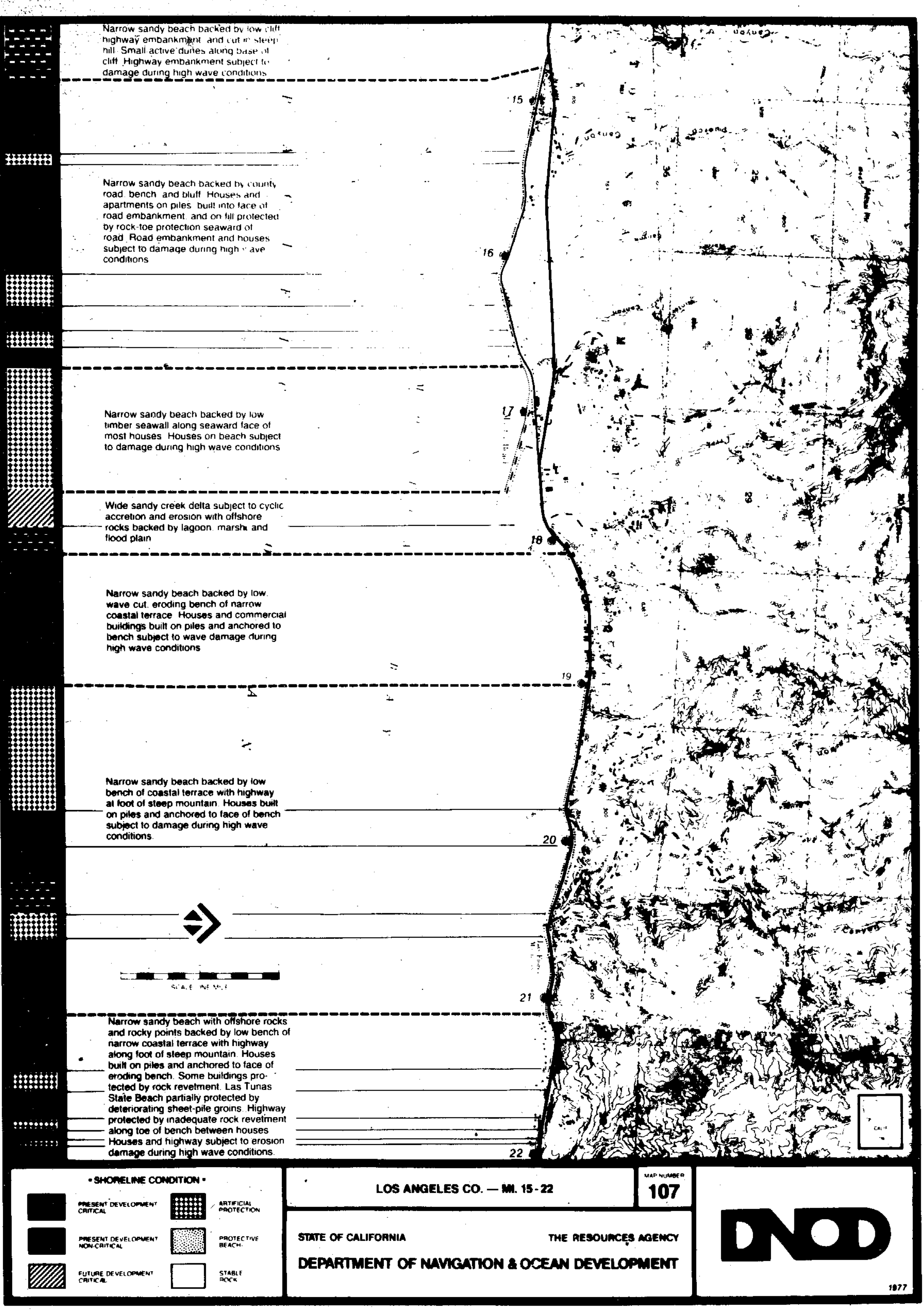


FIGURE 2 Map of coast showing potential winter storm damage.

FIGURE 3 View up coast toward Corral Beach from beach level at 25442 Malibu Road, March 8, 1978. The beach is stripped of sand and only cobbles remain. Mile 14.8.

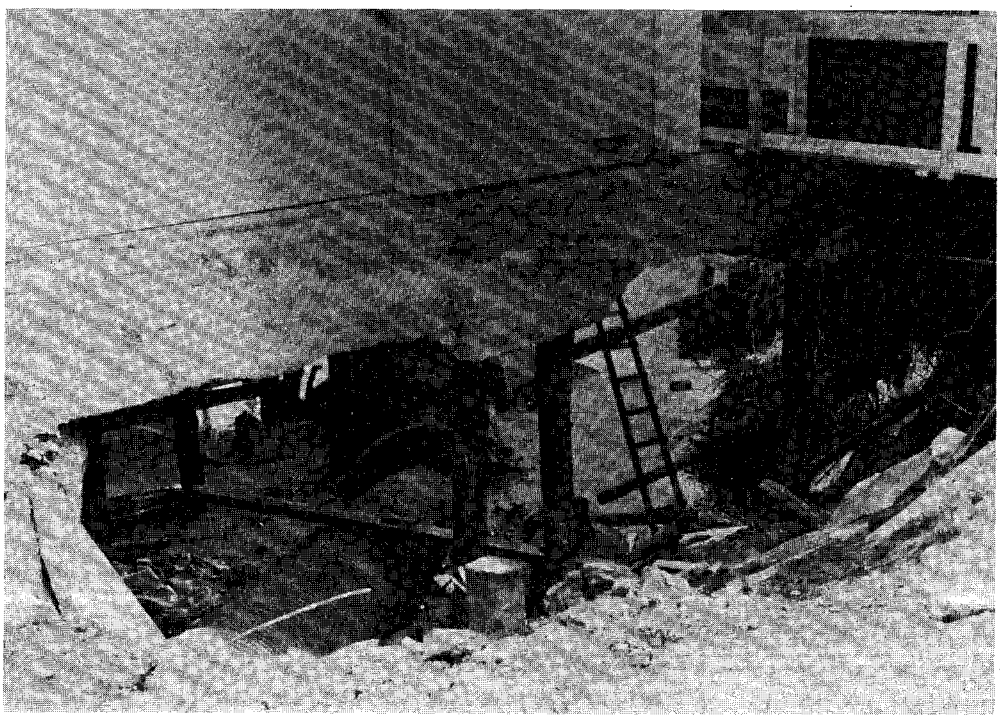


FIGURE 4 View seaward toward collapse of storm drain and slippage of driveway at 23950 Malibu Road, March 8, 1978. Mile 16.85.

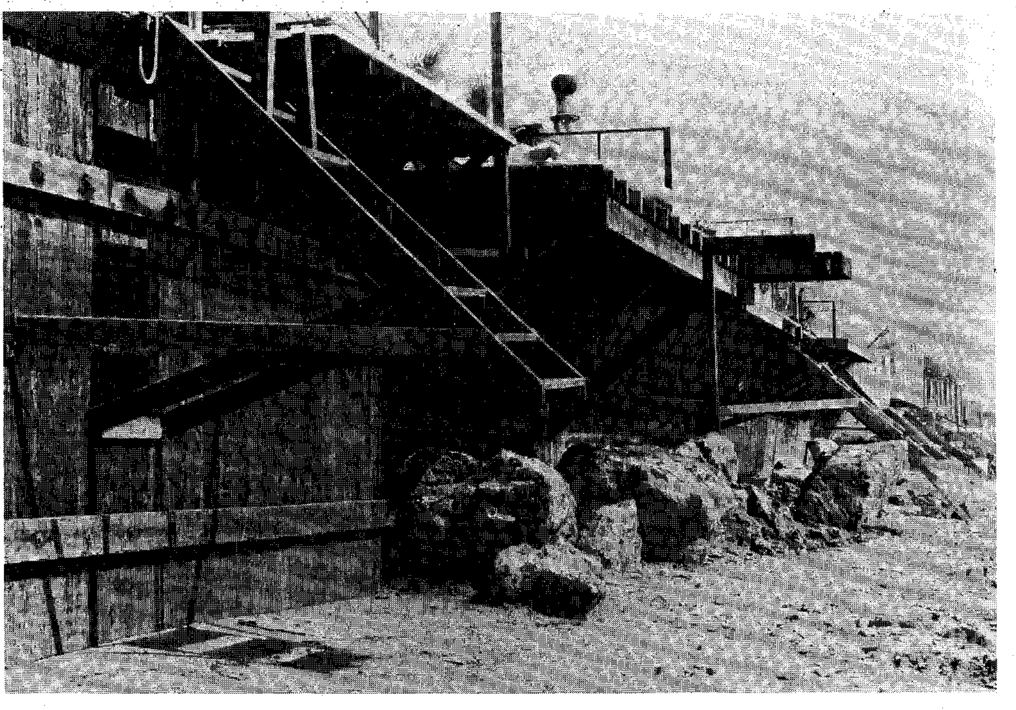


FIGURE 5 View down coast near lot 60 of Malibu Colony. Note depth of beach erosion below the top of the bulkhead and the emergency placement of rock rubble at the toe of the seawall.

FIGURE 6 View up coast of damaged patio and destroyed bulkhead at lot 42 of Malibu Colony, March 7, 1978. Mile 17.06.

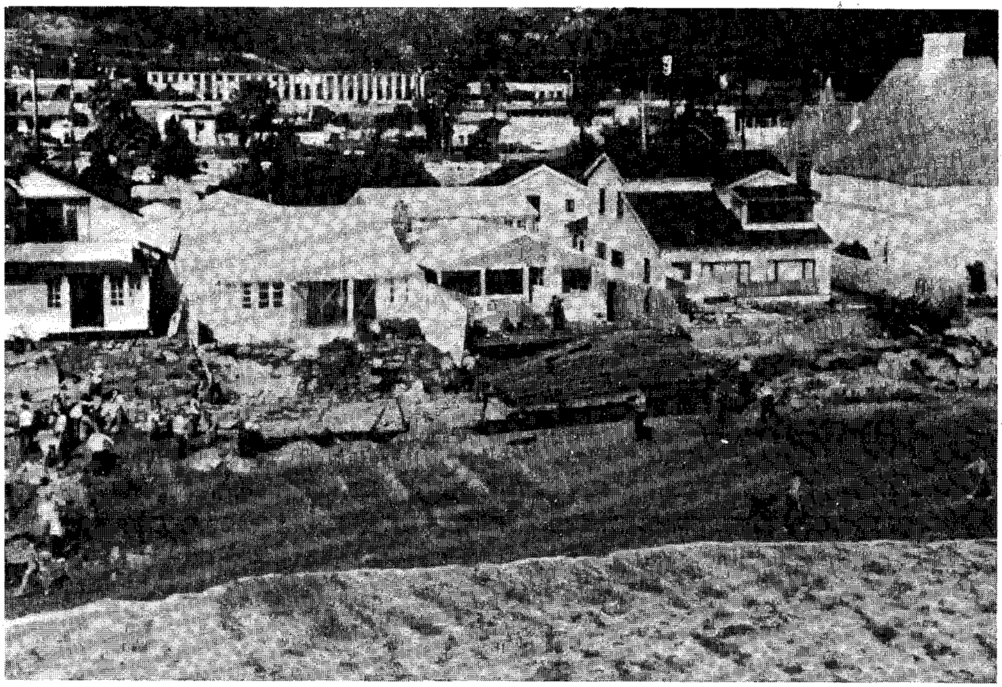


FIGURE 7 View toward land of beach area in the vicinity of lot 45 of Malibu Colony, March 3, 1978. Mile 17.1.

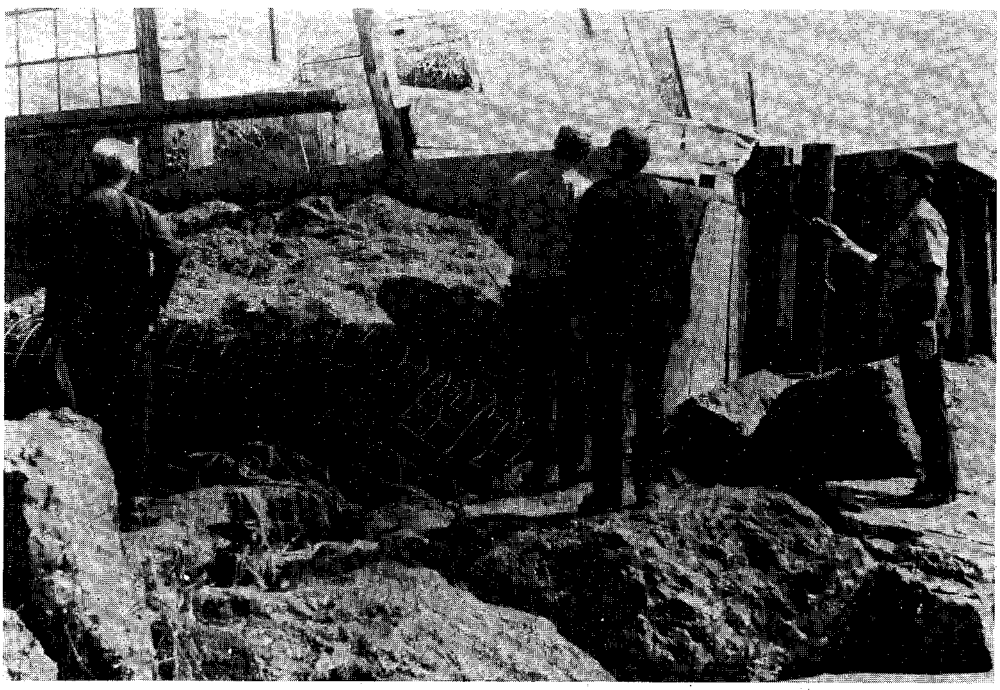


FIGURE 8 View of destroyed tea house at lot 45 of Malibu Colony, with a combination of pumpcrete and rock rubble for emergency protection, March 7, 1978. Mile 17.1.

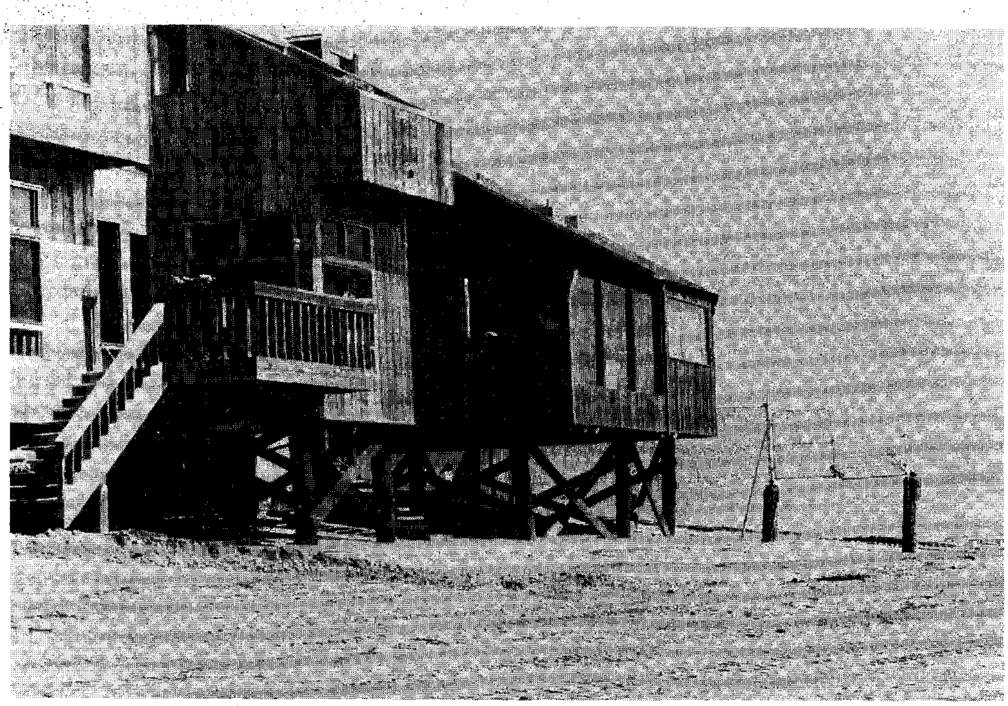


FIGURE 9 View down coast of beach from lot 116 of Malibu Colony, March 7, 1978. Note loss of beach beyond the boundary fence of the colony. Mile 17.55.

CONCLUSIONS

The cost of emergency work, repair, and loss of property resulting from the 1977-78 winter storms along the Los Angeles County shoreline in the vicinity of Malibu was about $4.0 million to private property alone. Coastal structures were damaged or destroyed and beaches were denuded of sand. Today all damage is repaired and the event is dim in the memories of most residents.

State-of-the-art predictive techniques in meteorology and oceanography cannot anticipate the extreme storm events that cause significant coastal erosion and accompanying damage to structures. The degree and/or rate of erosion during a storm is a function of overall climatic conditions and varies widely from storm to storm. The cumulative effects of tides, winds, and waves (or sea swells) play a major role in determining the erosion rate for a given storm.

The degree of shore protection depends largely upon the funds available for the construction of various devices. In the Malibu area, which is exclusively private property, individual landowners generally do not have the funds to design and build the needed protective structures that can withstand repeated attack by large high-energy waves. Therefore, during most emergency conditions, jury-rigged protection methods are used and do not provide permanent protection. As a general rule most homeowners devise their own emergency storm protection structures, which can either fail or cause accelerated erosion up or down the coast, thus affecting adjacent properties.

The Malibu area is located in a wave hazard area and can expect to be hit repeatedly by damaging waves in the future. The local residents need a substantial cooperative effort to protect their property. Los Angeles County officials met in the past with Malibu landowners and discussed the possibility of forming a shore protection district, but no action resulted from the discussion. Now that the major damage has been repaired or replaced, most landowners again feel secure, and thoughts of consolidating their efforts to upgrade their seawalls are lost again until the next major storm.

The 1977-78 storm season, which caused the worst shore erosion in the past 40 years, was exceptional but not unusual. A review of the storm conditions, the extent of wave damage, and the response of property owners and local, state, and federal agencies to these conditions has led the California Coastal Commission to draw the following conclusions.

1. Damaging storm waves can be expected to hit the California coast repeatedly in the future.
2. Damaged private development can be partially subsidized by local, state, and federal sources to repair or protect wave-damaged properties. Subsidies lessen the financial risk of developing in hazardous areas and may indirectly encourage further development in areas subject to wave attack.
3. Waivers of public liability are not an effective means of reducing the public subsidy involved in private property development.
4. Local governments and owners of existing coastal development are not prepared to handle damaging wave conditions.
5. There is substantial local, state, and federal expense involved in protecting and repairing public facilities that are vulnerable to wave attack.
6. The National Flood Insurance Program cannot adequately regulate development in wave hazard areas.

The U.S. Army Corps of Engineers suggested a joint effort by local, state, and federal agencies to develop and publish a brochure describing basic coastal processes and shore protection techniques for the various California coastal areas. This type of publication would make the public more aware of the potential hazards associated with living in the coastal zone and would provide some guidance in selecting alternative methods of shore protection. The brochure would also delineate emergency measures that could be initiated when a shore protection device fails and endangers an entire beach or shoreline. The damage at Malibu and other similar areas along the California coast during the 1977-78 storms identifies the basic shore protection needs for coastal residential areas: (1) A wide protective beach is the best protection against erosion and wave damage. (2) The best secondary defense against damage is a structurally sound seawall or bulkhead to resist high-breaking and overtopping waves. (3) Seawalls in areas with narrow protective beaches should be rockrevetted along the toe of the wall to prevent undercutting. (4) The seawalls must be maintained and replaced when necessary to withstand high-energy loads from large waves.

The Beach Erosion Branch of the Department of Boating and Waterways will continue to give technical assistance to local agencies and private landowners to upgrade their shore protection devices to reduce damage during future storms with the same magnitude as the 1977-78 events.

COASTAL RESPONSE OF LEADBETTER BEACH, SANTA BARBARA, TO SOUTHERN CALIFORNIA STORMS OF FEBRUARY 16-21, 1980

by Martha J. Shaw

The response of protective beaches to severe as well as mild wave conditions greatly determines the stability of coastal property and harbor channels along the southern California coastline. In February 1980 Pacific westerlies displaced southward generated a series of storms that caused unusually high waves at Santa Barbara, California. At this time, as part of the Nearshore Sediment Transport Study experiment, waves and the beach profile were being monitored at Leadbetter Beach in Santa Barbara. Within a period of several days the foreshore of Leadbetter Beach was drastically eroded and adjacent property was inundated and damaged. Beach surveys show that along a 709-m length of the beach approximately 79,000 cu m of sand was removed from the beach face to a depth of 2 m below mean sea level. Much of this sand was transported by waves to offshore bars and into the Santa Barbara Harbor.

INTRODUCTION

A series of storms originating several thousand kilometers west of southern California in mid-February 1980 generated large waves at sea. As the storms moved onshore they brought high winds and heavy precipitation to the southwestern region of the United States. The combination of high-energy waves, strong onshore winds, and a perigean spring tide caused severe erosion of beaches and extensive damage to adjacent property from direct wave impact

---

Martha J. Shaw is with the Shore Processes Laboratory of the Scripps Institution of Oceanography in La Jolla, California.

Note: Most of the data used in this report were collected in conjunction with the Nearshore Sediment Transport Study, which is sponsored by the National Sea Grant and the Office of Naval Research. Special thanks are extended to D. Inman, S. Jenkins, B. Waldorf, T. White, S. Pawka, M. Clark, M. Freilich, and G. Kuhn of the Shore Processes Laboratory, to R. Seymour of the Institute of Marine Resources, and to R. Dean of the University of Delaware for their contributions to the acquisition and processing of data appearing in this report.

and coastal flooding all along the southern California coast. A large-scale field experiment was under way at this time at Leadbetter Beach in Santa Barbara, California (see Figure 1), to monitor various physical parameters of the surf zone as part of the Nearshore Sediment Transport Study.

DATA COLLECTION

Beach profiles were surveyed at Leadbetter Beach along a minimum of three rangelines at least every two months in 1979 and monthly in 1980, with increased frequency during February, by the Shore Processes Laboratory. The beach profiles presented in this report were surveyed along rangelines SIO 1, SIO 2, and SIO 3 using standard rod and transit techniques to a depth of 1.5 to 2.0 m below mean sea level (MSL). The beach profile data were stored on magnetic tapes and processed on an Interdata Model 70 minicomputer to give profile plots and volume calculations. Fathometer data seaward of the three rangelines were obtained from the Department of Civil Engineering of the Univerity of Delaware.

Wave data were recorded by the west wave array of the California Coastal Engineering Data Network, which is sponsored by the California Division of Boating and Waterways. This array consists of four pressure sensors located approximately 362 m offshore of the SIO 2 benchmark at a depth of -9 m (MSL) (see Figure 2). Tidal data were obtained from tide gage measurements made by the National Ocean Survey at Stearns Wharf in Santa Barbara Harbor. Synoptic weather charts were obtained from the National Climatic Center.

WEATHER AND WAVE CONDITIONS

Typically, most of the winter swells reaching the southern California coast are generated by storms that originate in the vicinity of low-pressure centers in the Gulf of Alaska and thus approach from the northwest. These storm centers usually move out of the Gulf of Alaska and eastward well north of Point Conception, so that southern California has relatively mild weather. The refraction of northwest swells around Point Conception greatly reduces the wave energy reaching Santa Barbara. In addition to this limited exposure to northwest swells, Santa Barbara is partially protected from southerly swells, generated by tropical storms, by the sheltering effect of the Channel islands.

In mid-February 1980 low-pressure centers from the high latitudes migrated south over the central Pacific Ocean between 30°N and 40°N latitude. Due to a northward displacement of the subtropical jet stream at this time, warm moist air was entrained into these low-pressure cyclonic systems, causing intense disturbances that moved eastward (Jenkins, Shore Processes Laboratory, personal communication).

Waves of considerable fetch generated by these disturbances traveled eastward directly into the Santa Barbara Channel. Prior to the arrival of these long-fetch waves from the west, high-energy waves of shorter frequency from the south, generated by winds parallel to warm fronts, also hit Santa Barbara (see Figures 3a and 3b). Leadbetter Beach was thus subjected to

FIGURE 1 Leadbetter Beach, Santa Barbara, showing wide surf zone during the storm and inundation of the beach and adjacent property. (Shore Processes Laboratory photograph.)

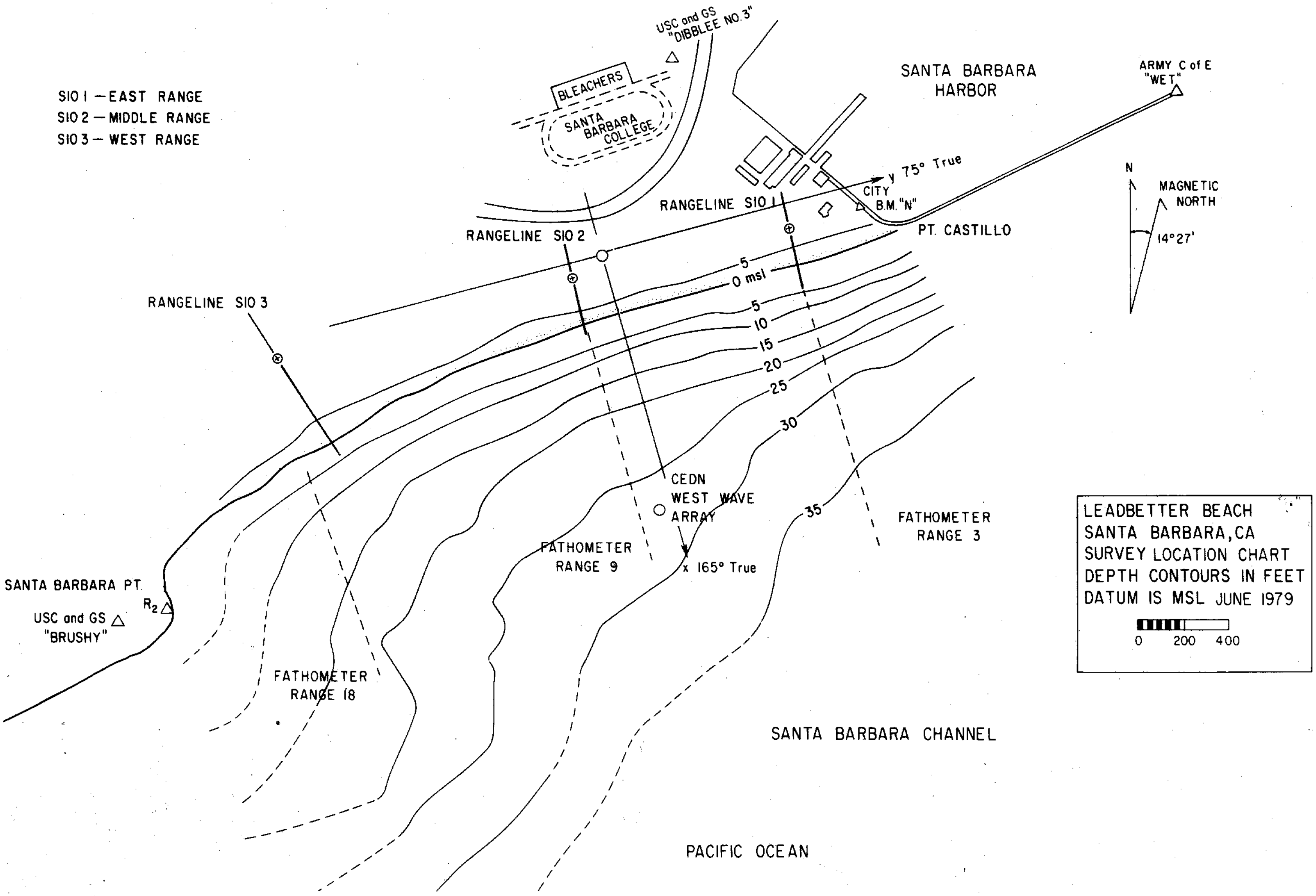


FIGURE 2 Map of Leadbetter Beach showing position of the beach and fathometer surveys and of the wave monitoring station.

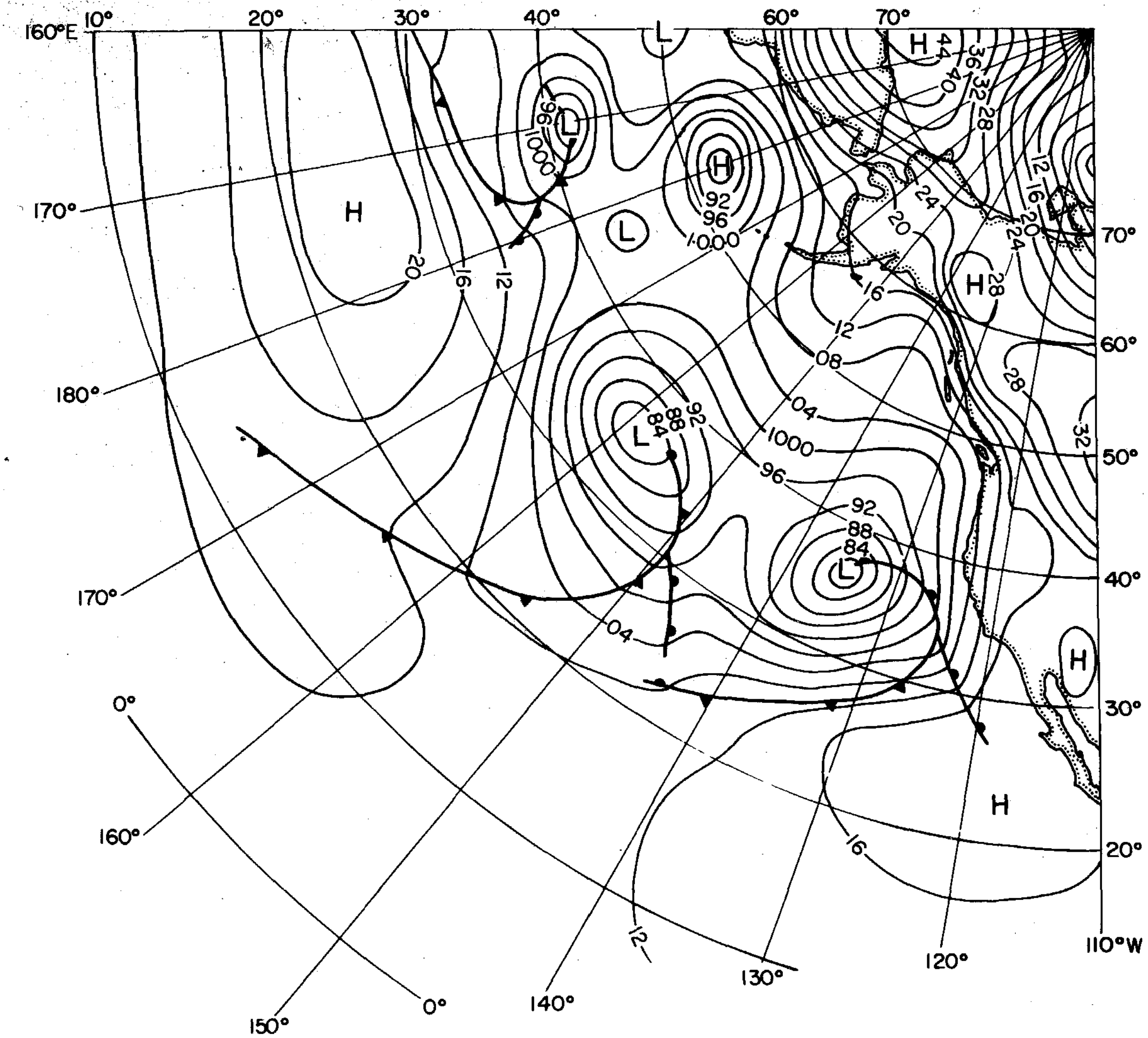


FIGURE 3a Surface synoptic weather map for February 16, 1980.

high-energy waves first from the south on December 16 and then directly from the west between December 17 and 21 (Figure 4). In Figure 5a wave energy is plotted for the month of February.

When waves approach obliquely there is a longshore directional component of wave energy (Scripps Institution of Oceanography, 1947; Komar and Inman, 1970). The longshore (y direction) component of the onshore (x direction) wave energy flux, Syx, can be expressed by the formula Syx = ECn(sin α)(cos α), where E is the wave energy density, Cn is the group velocity, and α is the horizontal angle of the wave front with the bottom contour. Syx, sampled four times a day at the west wave array, was normalized by dividing the total Syx of the wave spectrum at each sample time by the absolute maximum Syx for the month of February. This is plotted versus time

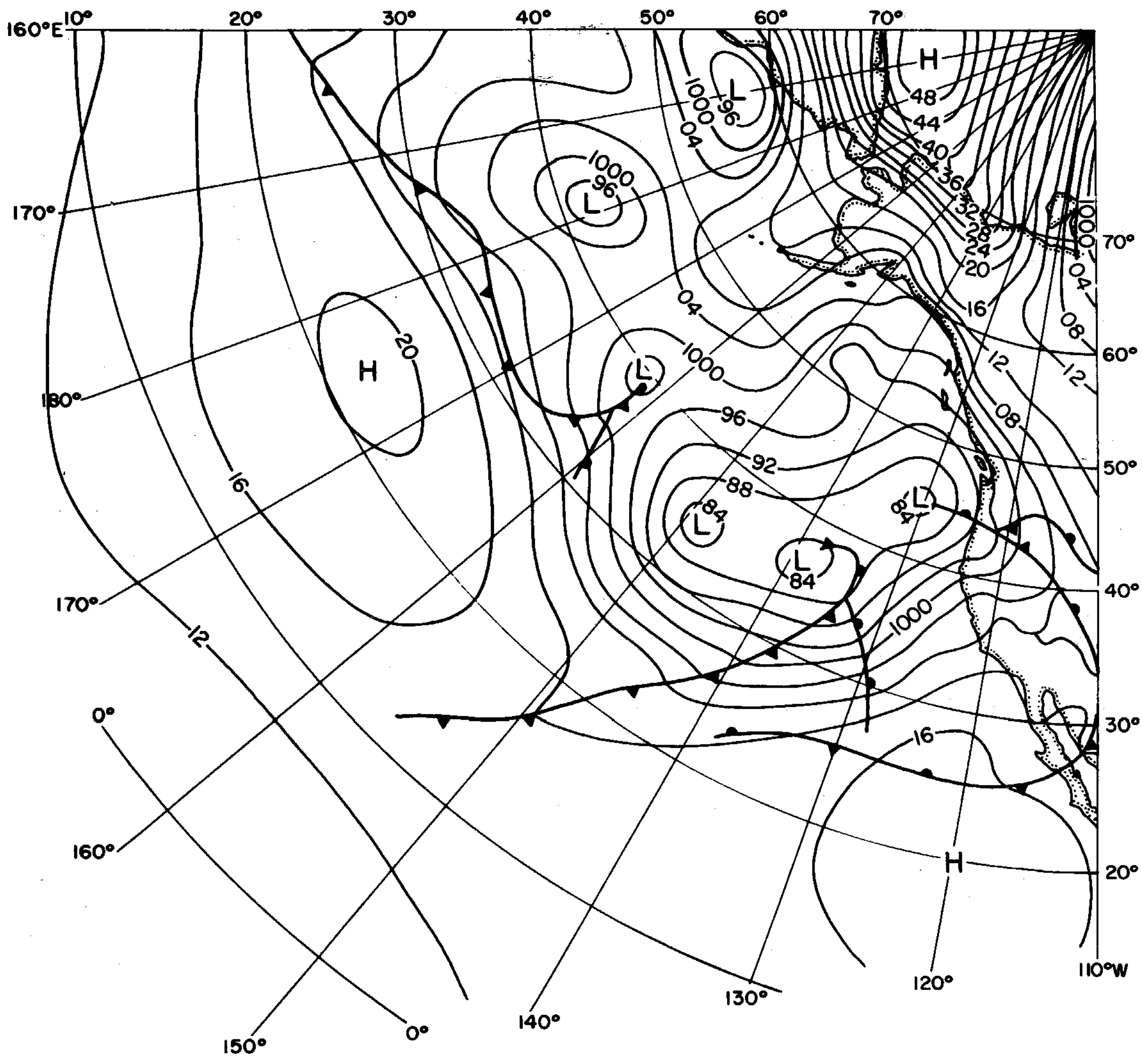


FIGURE 3b Surface synoptic weather map for February 17, 1980.

in Figure 5b. The orientation of the x and y axes are indicated in Figure 2: the positive x direction is eastward toward Santa Barbara Harbor, and the negative direction is westward toward Santa Barbara Point. As is apparent from Figures 5a and 5b, the dominant wave energy on February 16 was from the south, while on February 17-21 waves from the west dominated the total wave energy.

During the storm period, waves reached a height of at least 3 m. Figure 6 shows the magnitude of a breaker on February 19. The strong winds and heavy precipitation of the storms reached the coast almost simultaneously with these high-energy waves. Enhancing the destructive power of the waves at the study site was entrained seaweed, which entangled and destroyed many of the sensors installed in the surf zone for the experiment.

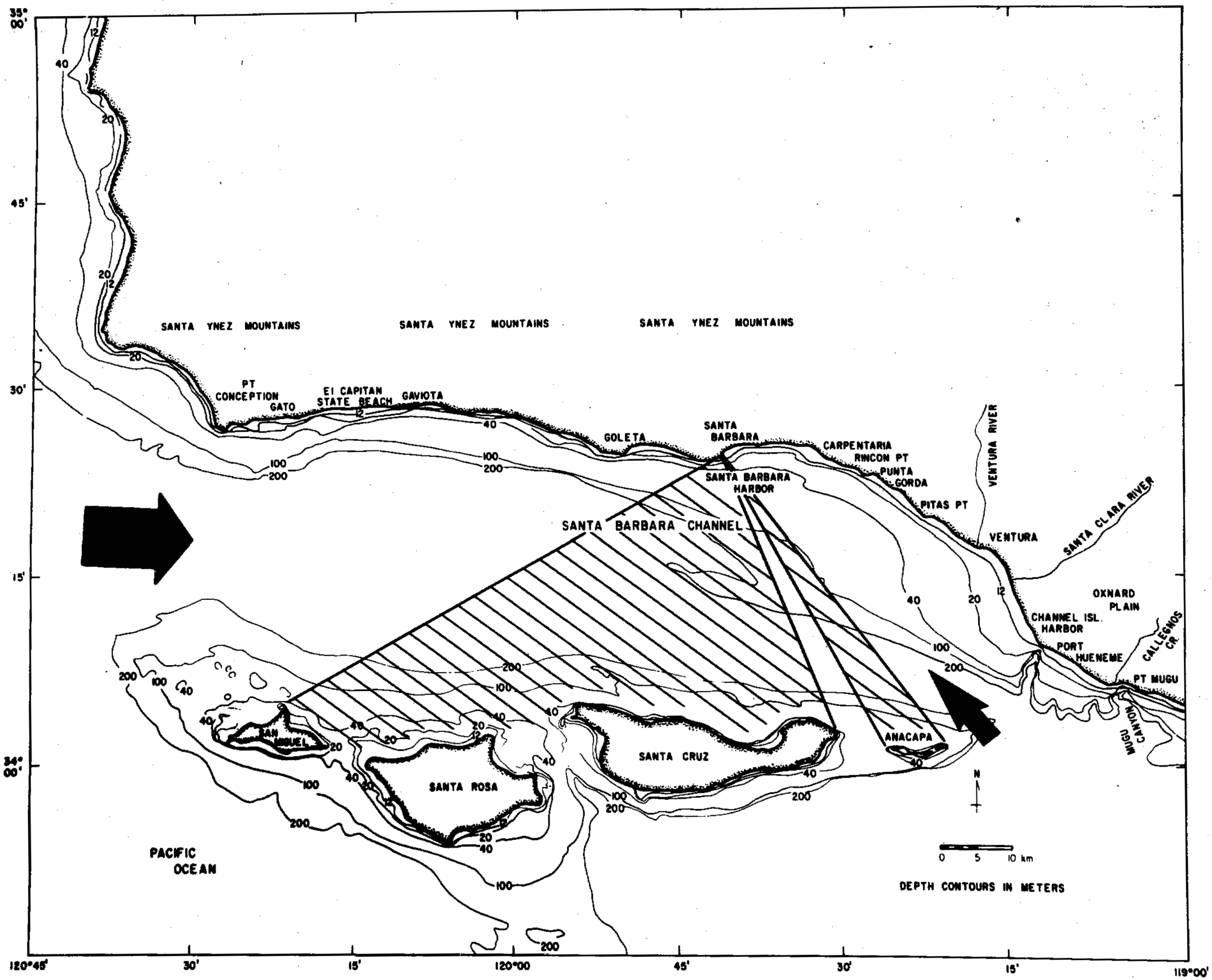


FIGURE 4 Location map of Santa Barbara, California, showing the wave shadowing effect of the islands. The arrows indicate the windows through which the storm waves of February 16-21 entered the Santa Barbara Channel.

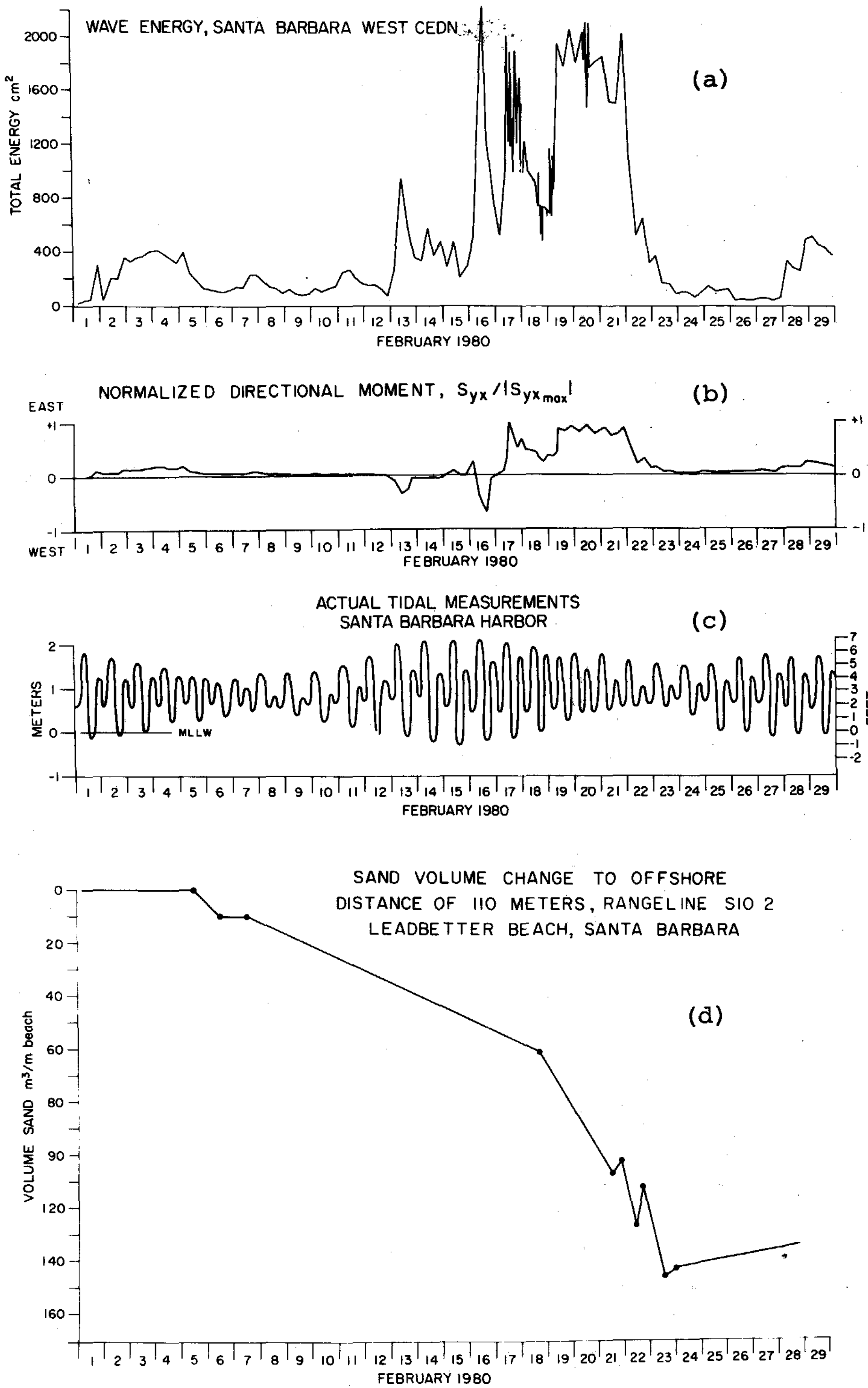


FIGURE 5 Wave energy, directional moment, tidal conditions, and the beach response at Leadbetter Beach in February 1980. (a) Total wave energy recorded four times a day at the west wave array. (b) Directional moment of the total wave energy normalized by the maximum for the month. The positive direction is east toward the harbor, the negative direction is west. (c) Actual tidal level measured at the National Ocean Survey tide gage in Santa Barbara Harbor. (d) Change in sand volume along rangeline SIO 2 to -2.0 m depth (MSL).

FIGURE 6 View of a breaking wave on February 19, 1980, at Leadbetter Beach. (Shore Processes Laboratory photograph.)

TIDES

Coincident with this storm was the occurrence on February 16 of a "perigean spring tide," an enhanced tidal amplitude resulting from the astronomical condition of perigee-syzygy (Wood, 1976) (see Figure 5c). Perigee is the position at which the moon is in its closest proximity to the earth in its monthly elliptical orbit; syzygy is the position at which the moon is in the same longitudinal plane as the earth and the sun (producing a full or new moon). The phenomenon of perigee occurring within 24 hours of syzygy occurs approximately three times a year.

The occurrence of perigee-syzygy with strong onshore winds and high waves has caused extensive coastal flooding throughout history. In this case syzygy, producing a new moon, occurred at 0100 PST on February 17, 1980. The enhanced tidal range, when combined with the waning beach profile and intense wave conditions, caused extensive inundation of lowland areas at high tides during the storm period. These unexpected conditions also forced evacuation of housing units and of many of the instruments being used by the Nearshore Sediment Transport Study (see Figure 7).

FIGURE 7 Inundation of the backshore of Leadbetter Beach at high tide on February 19, 1980. (Shore Processes Laboratory photograph.)

BEACH CHANGES

Figure 5d plots the change in sand volume versus time along rangeline SIO 2 during February for comparison with wave and tide conditions. The waves from the February storm rapidly removed sand from Leadbetter Beach. As the beach was undermined, adjacent sidewalks and other structures collapsed.

Profiles along rangeline SIO 2 during the first half of 1979 and the first half of 1980 (Figures 8a, 8b, and 8c) illustrate the relative stability of the beach's configuration throughout 1979 and January 1980, as well as the severe erosion of the profile in February (see Figure 9.) Between the surveys of February 8 and February 18 the mean sea level contour at rangeline SIO 2 retreated landward 12 m and approximately 51 cu m of sand per meter of beach were removed from the rangeline. The wave energy data indicate that this erosion began to occur on February 16. Between February 18 and 23 mean sea level retreated another 26 m and another 46 cubic meters of sand were removed,

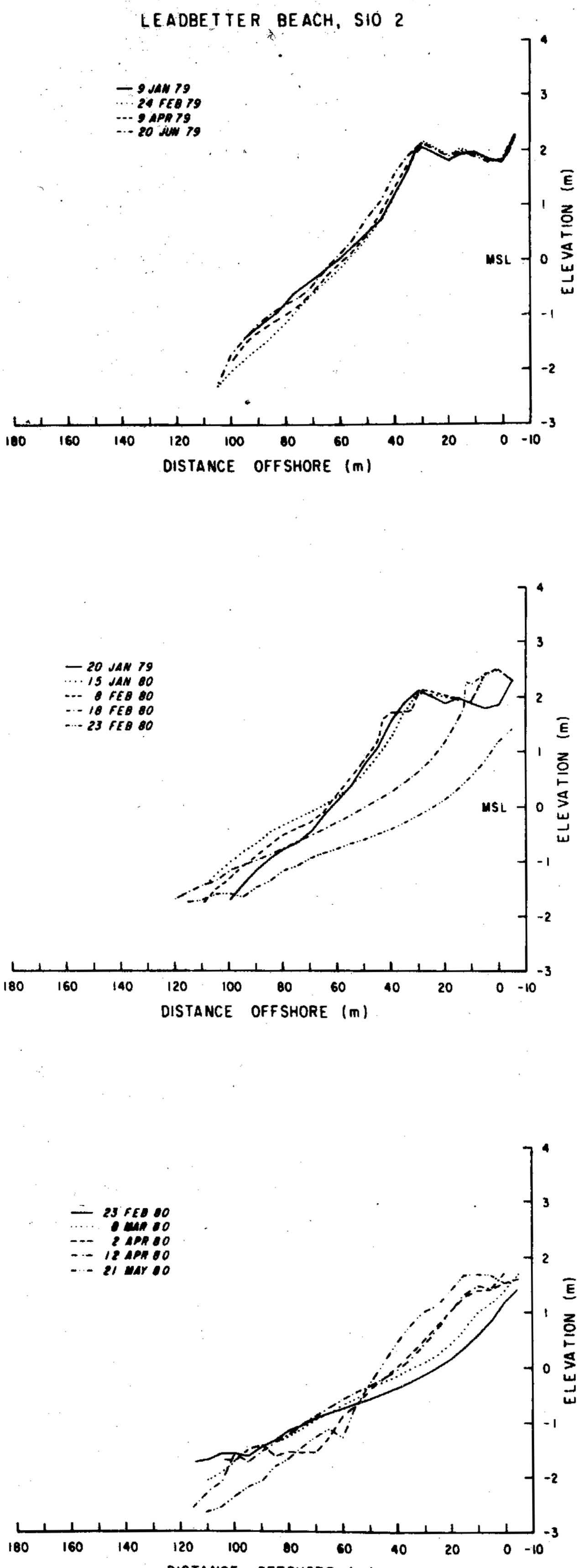


FIGURE 8 Comparison of beach profiles at rangeline SIO 2 from January 1979 to May 1980, showing the severe erosion by the February 1980 storm and the gradual rebuilding of the beach face after the storm.

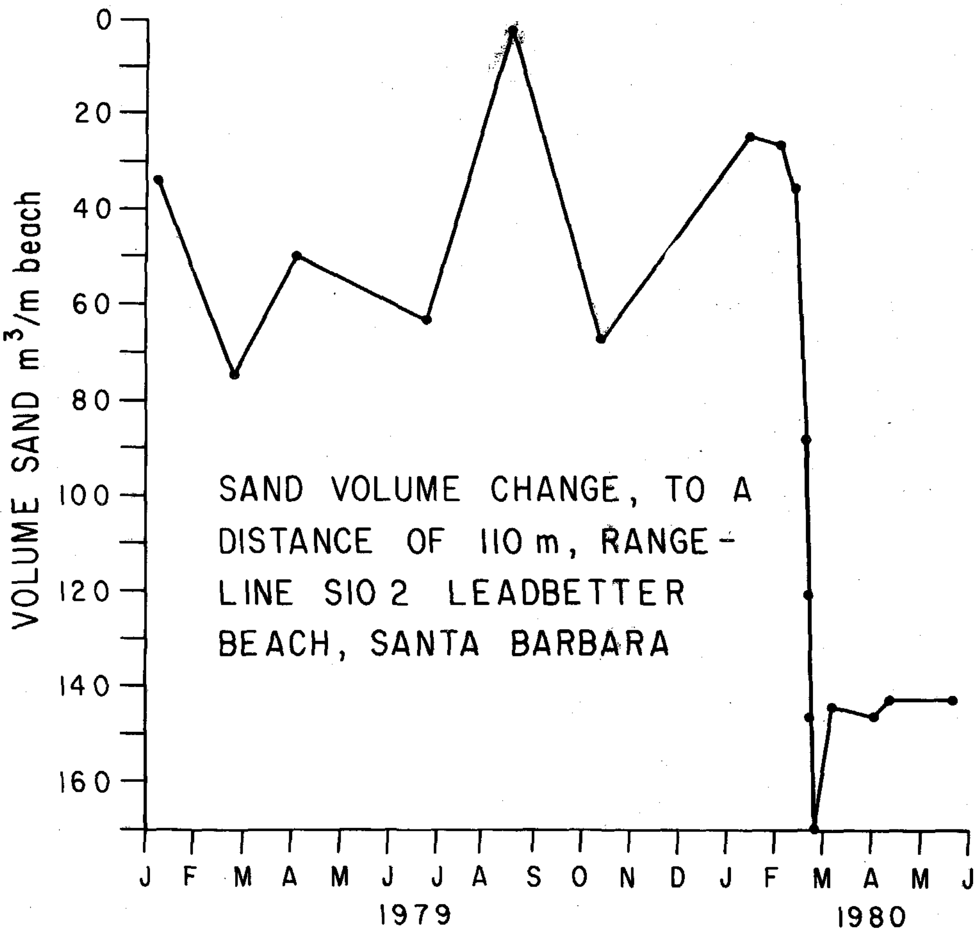


FIGURE 9 Change in sand volume along rangeline SIO 2 during 1979-80.

bringing the total retreat of mean sea level at this rangeline to 38 m and the removal of sand to 97 cu m.* Similarly, along rangelines SIO 1 and SIO 3 mean sea level retreated 32 and 46 m, respectively, with associated volume losses of 129 cu m of sand and 121 cu m of sand from the profiles. Extrapolation of the change in sand volume along the 709-m length of beach between SIO 1, SIO 2, and SIO 3 yields a total volume loss of 79,000 cu m of sand from the foreshore during the storm period. Poststorm surveys, presented in Figure 8, show the gradual restoration of the upper part of the beach profile throughout the spring. Also apparent in the poststorm surveys is the continued denudation at the seaward end of the rangelines, indicating an onshore movement of sand.

---

*This measurement refers to the change in volume within a 1-m-width of beachfront at the rangeline surveyed out to a depth of approximately -2.0 m (MSL).

High-energy storm waves typically remove large volumes of sand from the foreshore of beaches and transport it to offshore bars (Shepard, 1950; Inman and Filloux, 1960; Aubrey et al., 1980). Prestorm and poststorm beach profiles along rangelines SIO 1, SIO 2, and SIO 3 are plotted in Figure 10. At the seaward limit of the beach profiles, prestorm and poststorm fathometer surveys taken by the University of Delaware on January 22 and February 25 are superimposed as closely as possible, although the rangelines are not identical. It is apparent, however, that much of the sand removed from the beach above an approximate depth of -2 m (MSL) was transported offshore to depths between -2 and -7 m (MSL). Below -7 m the seafloor appears unaffected by the storm.

The Santa Barbara Harbor, built in 1931, typically traps sediment moving along the coast in the littoral zone (Inman and Frautschy, 1966). This necessitates an average dredging of 200,000 cu m of sediment per year to maintain navigation channels (Shaw, 1980). Shoaling, which causes waves to break in the vicinity of the entrance, is extremely hazardous to navigation. Rapid shoaling occurred in the harbor entrance between December 16 and 21, as shown by aerial photographs. Analyses of prestorm and poststorm fathometer surveys of the harbor indicate that 50,000 cu m of sediment accumulated in the vicinity of the harbor entrance (R. Dean, University of Delaware, personal communication). This substantial volume of sediment trapped by the harbor is evidence of the significant longshore component of sediment transport by the storm waves.

SUMMARY

Between February 16 and 21, 1980, a near alignment of the earth, moon, and sun coincident with unique global and local atmospheric conditions resulted in enhanced tidal amplitudes and intense waves along the southern California coast. The exposure of Santa Barbara, California, to long-fetch storm swells is limited due to the sheltering effect of Point Conception and the Channel islands. Powerful waves in mid-February, however, entered the Santa Barbara Channel directly through narrow south and west windows and removed large volumes of sand from the beaches. The rapid erosion of protective beaches left coastal property exposed to direct wave impact and tidal inundation (Figure 11).

Comparison of beach profiles taken at Leadbetter Beach in Santa Barbara throughout 1979 and the early part of 1980 show the drastic effect of this single climatic event on a relatively stable shoreline. Approximately 79,000 cu m of sand was eroded from a 709-m length of the beach to a depth of -2 m (MSL). Much of the sand removed from the beach was deposited offshore at depths of -2 to -7 meters (MSL). Sand was also transported longshore and into the Santa Barbara Harbor entrance. The longevity of this storm's effect on the shoreline is evident in poststorm surveys. By late May 1980 the beach had not yet been restored to its prestorm configuration.

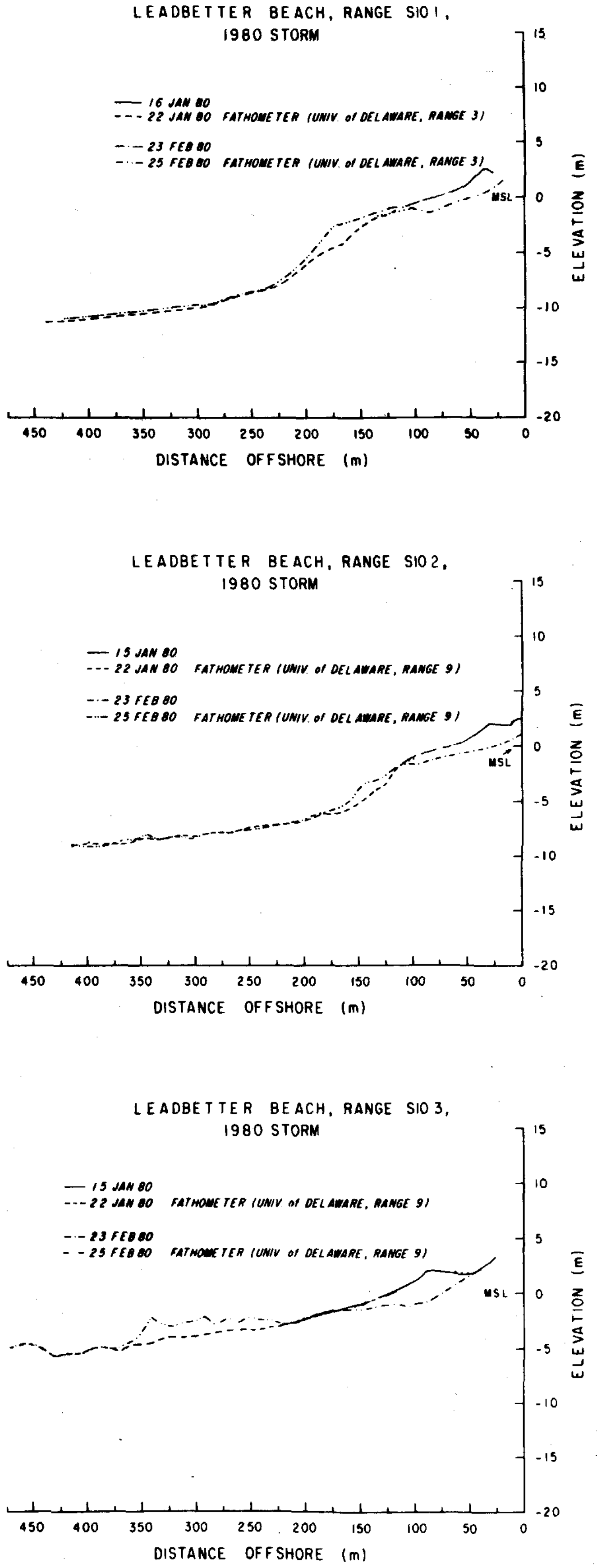


FIGURE 10 Prestorm and poststorm beach profiles at rangelines SIO 1, SIO 2, and SIO 3 with superimposed prestorm and poststorm fathometer surveys showing the development of offshore sand bars following erosion of the beach.

FIGURE 11 View of damage to Leadbetter Beach, February 23, 1980. (Shore Processes Laboratory photograph.)

REFERENCES

Aubrey, D. G., Inman, D. L., and Winant, C. D. (1980) The statistical prediction of beach changes in southern California, Journal of Geophysical Research 85(C6):3264-3276.

Dickson, R. R. (1980) Weather and circulation of February 1980, Monthly Weather Review 108:679-684.

Gable, C. G., ed. (1979) Report on Data from the Nearshore Sediment Transport Study Experiment at Torrey Pines Beach, California, Nov.-Dec. 1978, IMR Reference No. 79-8, Institute of Marine Resources, Scripps Institution of Oceanography, La Jolla, California.

Inman, D. L., and Filloux, J. (1960) Beach cycles related to tide and local wind wave regime, Journal of Geology 68(2):225-231.

Inman, D. L., and Frautschy, J. D. (1966) Littoral processes and the development of shorelines, Coastal Engineering, Santa Barbara Specialty Conference, American Society of Civil Engineers, Santa Barbara, California, pp. 511-536.

Institute of Marine Resources (1980) Monthly Report No. 51, California Coastal Engineering Data Network, Scripps Institution of Oceanography, La Jolla, California.

Komar, P. D. and Inman, D. L. (1970) Longshore sand transport on beaches, Journal of Geophyical Research 80(30):5914-5927.

Scripps Institution of Oceanography (1947) A statistical study of wave conditions at five locations along the southern California coast, University of California Wave Report No. 68, University of California, San Diego.

Shaw, M. J. (1980) Artificial sediment transport and structures in coastal southern California, Sediment Management Project for Southern California Mountains, Coastal Plains and Shoreline, SIO Reference No. 80-41, Scripps Institution of Oceanography, La Jolla, California.

Shepard, F. P. (1950) Beach Cycles in Southern California, Technical Memorandum 20, Beach Erosion Board, Corps of Engineers, Los Angeles.

Winant, C. D., Inman, D. L., and Nordstrom, C. E. (1975) Description of seasonal beach changes using empirical eigenfunctions, Journal of Geophysical Research 80(15):1979-1986.

Wood, F. J. (1976) The Strategic Role of Perigean Spring Tides, In Nautical History and North American Coastal Flooding, 1635-1976, Stock No. 003-017-00420-1, U.S. Government Printing Office, Washington, D.C.

PRESIDENTIALLY DECLARED MAJOR DISASTER FOR SEVEN SOUTHERN CALIFORNIA COUNTIES: THE JANUARY, FEBRUARY, AND MARCH 1980 STORMS

by Jacob Angel

This paper presents the experience of the writer as a member of a disaster survey team that assessed damages caused by the 1980 floods in southern California to facilities owned by local private nonprofit and public agencies. It explains how local agencies can improve the documentation of damages to receive financial assistance under Public Law 93-288 and how they must mitigate for future flood damages.

The paper describes the makeup of a disaster survey team (which includes a representative from the federal government, the state government (which the writer represented), and the local agency); the overall extent of damages; the type of damage incurred; the Damage Survey Reports (DSRs) submitted by disaster survey teams as required by the Federal Emergency Management Agency for each category of damage incurred; the information needed to complete the DSRs; the measures eligible public and private agencies can take to improve documentation of future flood disaster claims; the percentage of funds local agencies are reimbursed depending on the type of repair work to be done; the writer's opinion as to the cause of damage; and the mitigation of flood hazards required of each agency by relocating, flood proofing, flood insurance, or construction and operation of flood control facilities.

The January and February 1980 storms wrought widespread damage and destruction in southern California. After the storms were over and the floodwaters had receded, requests for governmental financial assistance were made to repair the damages.

Based on preliminary data provided to the California Office of Emergency Services (OES), the storms in southern California resulted in 18 deaths, 145 injuries, 1,344 homes damaged, 111 homes destroyed, and 282 businesses damaged. Water control facilities, public buildings and utilities, road systems, and agricultural lands also were damaged. The initial flood-related damage dollar estimates were \$267 million (\$155 million to public facilities, \$69 million to private property, and \$43 million to agriculture). As a result of these damage estimates, Governor Brown declared a state of emergency in

---

Jacob Angel is Senior Engineer with the California Department of Water Resources in Sacramento, California.

seven southern California counties and later in three northern counties. In response to a request by the Governor the President declared eight counties in California a major disaster area, which allows state and local government agencies to file applications seeking federal funds under Public Law 93-288 to be used in alleviating the effects of the storms.

Public Law 93-288 provides for local agencies to be reimbursed up to 100 percent of the direct cost of disaster-related work. It allows for the permanent restoration of facilities to their predisaster condition in accordance with the codes, specifications, and standards in effect at the time of the disaster. In this past disaster the Federal Emergency Management Agency (FEMA) also indicated that it would allow up to 10 percent for disaster proofing to alleviate or prevent the recurrence of damage to facilities located in flood hazard areas, if those facilities could not reasonably be relocated from the floodplain. Many projects, however, will require major changes to implement hazard mitigation measures. These are costs that may have to be borne mainly by the local jurisdiction.

After the declaration by the President, 73 damage survey teams were dispatched to 430 local agencies in seven southern California counties and one northern county to prepare Damage Survey Reports (DSRs) for each disaster-related project. These reports, submitted to FEMA and coordinated with OES, serve as the basis for federal funding to restore facilities and repair damages caused by the 1980 floods.

I was a member of one of the damage survey teams. Each damage survey team consisted of an engineer from a state agency and a federal agency accompanied by a representative of the local agency who identified the specific damage project.

As of June 12, 1980, 8,000 DSRs had been submitted for emergency and permanent work at public and private nonprofit facilities under the nine categories of work shown in Table 1. You should note that the DSRs are for public and certain private nonprofit facilities, which are the only ones eligible under Public Law 93-288. These DSRs for $113 million were less than the $155 million preliminary estimates submitted to OES, partly because the original estimates were too high, and partly because some projects were not available for review and cost factors were based on local rates rather than FEMA's rates. I want to stress to representatives of public agencies the importance of good documentation of unit costs and quantity for each facility repaired. Not having this information available for the damage survey team at the time of inspection delays the preparation of DSRs and the approval of reimbursements to eligible local agencies.

From my own observations as a member of a damage survey team, I came to believe that the major flood emergency and restoration work involved debris deposition, erosion of roads, and distruction of utility lines in the bottoms of channels. I was pleased to see, based on the tabulation of DSRs to FEMA, that my field experience covered the major types of projects.

The required contents of DSRs do not, in my opinion, place an unreasonable burden on applicants. The DSR forms ask for the following information.

TABLE 1 Categories of Public and Private Nonprofit Facilities Eligible for Governmental Assistance (arranged in order of dollar damage)

Category of Work	Damage (millions of dollars)
1. Debris Clearance	33.7
2. Road Systems	31.0
3. Water Control Facilities	19.0
4. Public Utilities	13.8
5. Protective Measures	10.0
6. Other Damages (not included in other categories)	3.8
7. Facilities Under Construction	1.0
8. Public Buildings and Equipment	0.6
9. Private Nonprofit Facilities	0.1
Total	113.0

- The name of the applicant
- The location, identification, and description of the damaged facilities--with vicinity and location maps and plan views of the damaged area
- A description of the damage
- Pictures showing the damaged areas
- The scope of proposed repair or restoration work
- Classification of work in the previously shown categories
- Whether the work is to be by contract or force account
- The percentage of work completed at the time of inspection
- Estimated quantities verified by field inspection
- Unit prices of labor, materials, and equipment, and estimated production rates

Despite the simplicity of the requested information, a substantial number of applicants did not have the required information ready when the damage survey team arrived at the damage site. This slowed down the whole process. Fortunately, a majority of the applicants (both large and small agencies) had the required information available at the time of the site inspection. It was obvious that those agencies had planned ahead and established procedures that readily segregated the costs of labor, materials, and equipment used during the flood fight from the costs of subsequent repairs and restoration at each facility. For some of the agencies a lack of adequate documentation reduced the amount found eligible by the damage survey teams or the review process.

When the DSRs were completed by the damage survey teams, they were sent to FEMA to be reviewed for completeness, accuracy, and engineering comments. After FEMA's review, copies of the DSRs were transmitted to OES for preparation of a formal project application and for a state analysis, which is in turn submitted to FEMA for review and approval. Applicants may, upon this submittal of their applications, request an advance of up to 75 percent of the recommended funding.

Under the President's executive order and federal disaster statute, DSRs have been reviewed to determine whether or not certain facilities could reasonably be moved out of the floodplain to avoid repeated damage. Those facilities that may not reasonably be moved are identified for possible disaster proofing to minimize future damages. It should be emphasized that FEMA has become stricter in requiring flood mitigation or flood proofing to minimize repeated damages before approving submitted claims. This is required by statute. In general, the damage survey teams submitted requests for repairs only if there was sufficient mitigation or flood proofing to prevent or reduce a repetition of flood damages in the future.

Originally, I had planned to provide representative examples of flood damages and the actions that might be taken to eliminate or reduce future damages. Instead, I suggest that you read the Department of Water Resources' pamphlet Reducing Flood Damage (see the appendix of the following paper, "New Approaches to Flood Hazard Mitigation," by Ronald B. Robie). The pamphlet provides much more detail than I could provide. Even more detail can be found in Department of Water Resources (1980).

As an employee of the Department of Water Resources I must, in closing, convey several strong, clear messages to the governmental structure in southern California. First, while some of the damage in 1980 was due to an "unusual event," most of the damage could be attributed to a lack of foresight in permit, design, or construction processes. Second, these mistakes of the past will be largely eliminated when local agencies initiate and carry out strong programs of floodplain management and zoning and hazard mitigation. Third, these programs must not only permit flood control agencies to become involved in the planning, zoning, and building permit process but must require their active participation in that process. Fourth, the flood control agencies must broaden their horizons beyond structural measures and consider all the actions that might be taken to reduce future flood damages.

Finally, the floods of 1978 and 1980 have clearly demonstrated the need to implement floodplain management techniques (and do preventative planning) before opportunities for low-cost solutions are lost. The representatives of flood control districts must today assume a strong leadership role to reduce the flood hazards of tomorrow. Many flood control districts in California have, in recent years, moved aggressively forward with flood damage prevention programs that proved their worth in 1980. We encourage such efforts and hope that all flood control districts eventually will have active damage prevention programs.

REFERENCE

Department of Water Resources (1980) California Flood Management: An Evaluation of Flood Damage Prevention Programs, Bulletin 199, Department of Water Resources, Sacramento.

NEW APPROACHES TO FLOOD HAZARD MITIGATION

by Ronald B. Robie

The winter flooding in California during 1980 resulted in the nation's taxpayers spending needless dollars on disaster assistance and in many of California's citizens suffering personal hardships. To a large extent, damage occurred because of the reluctance of local government to keep people and their property from creating disaster potentials. Flood damage in the nine-county disaster area was largely the result of human carelessness or ignorance in the form of inappropriate activities encroaching onto floodplains.

The Disaster Relief Act of 1974, as amended, is the most recent program that provides incentives to state and local governments to identify natural hazards, evaluate existing protective measures, and develop and implement hazard mitigation plans. The ideal flood hazard mitigation plan would seek to keep new development out of the floodplain and relocate existing damageable improvements. Practical economic and political considerations tend to preclude total achievement of this ideal; however, recent state and federal financial sanctions and incentives discussed in this paper hold promise for inducing local governments to restrict the use of their flood-prone areas.

The financial sanctions and incentives discussed emphasize that it is appropriate and prudent to require aid recipients to take reasonable precautions against foreseeable future damage.

INTRODUCTION

After several years of work the Department of Water Resources has recently released <u>California Flood Management: An Evaluation of Flood Damage Prevention Programs</u> (Bulletin 199). This bulletin discusses historical patterns of flood control and new directions recommended by the department. Of particular importance to water planners are maps of the entire state, basin by basin, showing existing flood control projects and flood hazard areas. Structural flood protection projects and nonstructural flood management plans in each hydrologic area are also described. Exemplary projects are described and depicted.

Ronald B. Robie is Director of the California Department of Water Resources in Sacramento, California.

Along with the bulletin the department has prepared a brochure describing many ways of reducing flood damage, using specific problems in Los Angeles County as examples. This brochure is included as an appendix at the end of this paper.

TRENDS IN FLOOD MANAGEMENT

Once, flood-prone areas were developed with the recognition that occasional floods were inevitable. However, because federal disaster assistance funds were available, some flood-damaged structures have been rebuilt in the same places, only to be flooded again.

California has seen much unwise development in floodplains. Mission Valley in San Diego is a prime example, as the area's continuing problems during winter rains demonstrate.

Over the last 60 years, flood protection projects have been built on many of the state's major rivers. A few locations, as part of an overall flood management program, still need major structural projects.

In considering flood management options, it must be realized that no project can provide absolute flood protection. Projects are designed to mitigate damage from floods of a specific magnitude, such as the 50-year or 100-year flood.

In addition, the original level of protection can be lowered by unforeseen watershed changes, such as intensive urban development, or by changed hydrologic assumptions, such as the recognition that storms greater than previously recorded could occur. The Santa Ana River is a classic example. Changed hydrologic assumptions and increasing upstream development have rendered the existing Prado Dam inadequate to protect the substantial downstream development, yet the floodplain continues to be developed at a rapid rate. Construction of physical works, along with many of the actions discussed later, is the only alternative, and will substantially mitigate flood hazards.

This project, now being considered by the Congress--and supported by the state--demonstrates a couple of realities of the 1980s. Such projects are extraordinarily expensive. The latest estimate for the Santa Ana project is $865 million. Also, the local costs of lands, easements, and rights of way, traditionally paid mostly by the state, are also high and cannot all be assumed by the state. The local area to be protected, with improvements valued at $34.3 billion, is capable of paying a larger share of this burden, considering the enormous benefits it will receive.

In such cases, where assumptions or situations have changed, people in the floodplain may have a false sense of security and fail to realize that a large flood can do severe, unexpected damage.

Local communities with serious flood problems that exceed the local capacity of funding traditionally turn to the federal government, and particularly to the U.S. Army Corps of Engineers, for help. Approval of a federal project usually leads to federal funds being appropriated to build the project and to considerable state financial aid for the project; this relieves local residents of much of the project's costs.

In the past most federal projects have been structural solutions. However, the Corps has now developed procedures to comply with the Water Resources Development Act of 1974, which requires that nonstructural alternatives be considered. We encourage local agencies to recommend these nonstructural solutions. We believe that the Water Code should be amended so that state financial aid for nonfederal costs will include costs for nonstructural measures required by the federal law.

Because federal funds for flood investigations and project construction are limited, local agencies compete for the money. Success is not necessarily related to the merits of a particular project. The California Water Commission has worked long and hard to evaluate projects proposed in the state and see that the money is wisely spent in the areas of greatest need and concern. To give priority to the more critical flood problems, the Department of Water Resources has begun an objective evaluation of the investigations and projects, based on their relative contributions to reducing flood damage in the state. We are then recommending to the Congress priorities for studies and projects that the Congress is considering for federal authorization or funding. The days of supporting all projects, regardless of merit, are over.

Changing public attitudes about flood control and governmental spending are bringing about new approaches to flood management. Although some in local government (and even in the flood control field) still mainly emphasize structural methods to prevent flood damage, it is generally recognized that the structural solution is only one of the options to be considered in planning flood management.

In some cases, additional physical works should be built, complemented by programs to deal with the remaining flood risk. In others, extensive programs of floodplain management should mix structural and nonstructural measures as needed. Nonstructural measures include floodproofing, flood warning, watershed treatment, and removal or relocation of existing developments from the floodplain. Together with an emphasis on nonstructural solutions, state and federal laws have been enacted to restrict the use of flood-prone areas and to mitigate existing hazards.

REGULATORY CHANGES

In 1968 Congress enacted the National Flood Insurance Program. This is an extremely important law. The Flood Disaster Protection Act of 1973 modified the original act and made it a major element in floodplain management. When a community participates in the program, flood insurance becomes available to its members, and participating communities must adopt plans for floodplain regulation. Prodded by the Department of Water Resources, most flood-prone California communities have entered the program to carry out the floodplain regulations it requires.

This law emphasizes keeping people from floods rather than the more expensive and traditional keeping floods from people. Many attack and try to thwart this law because it takes a regulatory approach and does not represent "business as usual." Yet prior to the National Flood Insurance Program, nonstructural flood management had not been widely practiced in California or elsewhere. This was due to opposition by landowners and development interests, local pressure for growth and increased tax bases, and the reluctance of local governments to resist these pressures.

The Federal Emergency Management Agency (FEMA), in response to statute, has imposed regulations requiring that natural hazards be evaluated and mitigated by all levels of government and that mitigation of natural hazards be enforced as a condition for obtaining federal disaster assistance. This should complement the National Flood Insurance Program. Unfortunately, this concept of hazard mitigation sounds better in the regulations than it does in real life. The main sanction--withholding funds--only comes <u>after</u> a disaster, when a community is digging out of the rubble of a flood. FEMA is unlikely to deny relief in the absence of a mitigation plan. Doing so would be criticized as the height of bureaucratic nit-picking, and the political pressure to rescue people, no matter how improvidently they have acted, is great. Yet once the damage is repaired, what is the incentive to develop a mitigation plan? It depends on the circumstances, but, clearly, hazard mitigation can only be meaningful when carried out <u>before</u> disasters.

Our department is committed to ensuring that natural hazard mitigation measures are effectively carried out as a condition of federal and state disaster assistance. For example, with regard to the 1980 floods in the Sacramento-San Joaquin Delta we are pushing for hazard mitigation--in this case, improvements of the terribly maintained levees that failed. It is an uphill struggle because certain governmental mechanisms, such as reclamation and levee districts, serve to insulate the owners of flood-prone land from both legal and social liability and responsibility. The public subsidies that result are significant. For example, Webb Tract (which was flooded and is now being reclaimed) has been valued at $5.4 million. Levee repairs and other restoration after this year's floods are costing the nation's taxpayers approximately $17 million. The district has spent about $100,000 for this work, and its annual expenditures for levee maintenance averaged only $82,000 since 1973. The island has 15 landowners, yet they have no personal liability. A virtually insolvent reclamation district owns the levees and is the only one held accountable. The burden has been shifted from the shoulders of the landowners. The hazard mitigation plan proposed by the Department of Water Resources would require levee improvements in the next three years costing $2.4 million. The district would pay this sum but could be reimbursed by almost $1 million from the Office of Emergency Services.

The state and FEMA should also develop more meaningful sanctions against local governments that allow unsuitable developments in floodplains. As a start, a cooperative effort is being proposed between the Department of Water Resources and FEMA to monitor compliance by California communities with regulations of the National Flood Insurance Program. Department officials will meet with community officials over a two-year period to discuss wise use of floodplains, to evaluate how each city and county is complying with the flood insurance program's building permit and variance procedures, and to see if governments are guiding development out of the floodway and, where possible, away from the floodplain. The project will also document any innovations by a particular community that might be useful to others.

An ongoing program of great significance is the Designated Floodway Program of the State Reclamation Board. Thus far, 1,100 miles have been protected under this program in the Central Valley. The program is described in Bulletin 199.

Also, the department is now evaluating flood hazards in its review of environmental impact reports to determine if a proposed development is subject to flooding. This evaluation will allow mitigation of flood problems early in the planning process, and will possibly set aside floodplains for use as open space.

MITIGATING FLOOD HAZARDS

The department recommends that all future state appropriations for flood disaster relief include requirements to mitigate hazards. The first step was taken with the passage of Senate Bill 366 in 1979, which affects Los Angeles and Riverside counties and the City of Los Angeles. This law requires that adequate land use controls be applied to make sure that new construction or rebuilding in areas of flood or debris hazard is allowed only where adequate protection is provided.

To comply with this law, we are asking the governments of both counties to pass ordinances requiring that actions relating to flood hazard identification, prevention, or mitigation be taken after consultation with county flood control districts; reimbursement of up to $3.3 million would act as an incentive to this consultation. The ordinances also seek to have the districts review, in flood-susceptible areas, subdivision maps, building and grading permits, flood zoning, the acceptance of storm drains dedicated by developers, the setting of water surface elevations, and the delineation of areas where building is not permitted. The appropriate decision-making or executive agency would prepare written findings regarding flood hazards after reviewing the districts' reports. The ordinances would also require that complete and accessible data files be maintained. Riverside County has already passed such an ordinance and has further delegated some approval authority in these areas to the county flood control district and the Coachella Valley Water District.

We have asked the City of Los Angeles to consult with the Los Angeles County Flood Control District on major projects that could overload district flood and drainage facilities as a condition of receiving reimbursement of up to $900,000. We believe that the agency responsible for effective operation and maintenance of drainage facilities should be involved as a consultant in the process of approving development. We are encouraging the City of Los Angeles and Los Angeles County to adopt ordinances similar to those adopted by Riverside County.

Another reason for inadequate flood-hazard mitigation has been the lack of an adequate base of technical information. FEMA needs to speed up technical studies of flood risks to provide a basis for floodplain regulation and flood insurance. The state could contribute more to these studies, which when complete will leave no excuse for planning, zoning, and public works agencies to continue to ignore nonstructural flood management in undeveloped or partly developed areas. This, of course, must be done <u>before</u> further development makes such management impractical.

It should be obvious that a flood hazard mitigation project is effective only if access and funding for proper maintenance are provided. With federal projects the price the local government pays for accepting federal funding includes paying for perpetual maintenance to preserve the capacity of the project. Yet the type of maintenance that has been provided for the channels and levees in some areas leaves much to be desired. In addition, maintaining agencies often in recent years have paid little heed to environmental considerations. There is public support for greater environmental efforts, however. Maintenance practices must be modified considerably to reflect environmental and esthetic values better. Bulletin 199 documents successful efforts and should be helpful in this area.

Greater recognition is being given today to the value of wetlands and riparian vegetation--not only for ecological, recreational, and esthetic reasons but also for reasons of flood mitigation and water quality. Some counties have passed ordinances to protect riparian vegetation, and state agencies are encouraging the concept.

It is state and federal policy not only to preserve but also to enhance the wetlands of the state and nation. The State Department of Fish and Game requires a permit before a streambed can be altered. The State Water Resources Control Board, in its Decision 1460, ruled that diverting nonflood flows of a stream segment that furnishes vegetative habitat is a waste of water and unreasonable. This permit requirement and ruling are intended to keep development from eliminating stream segments or from channelizing them without mitigation.

All agencies involved in planning, zoning, and public works need to emphasize protection of wetlands and riparian vegetation as a technique to manage floods nonstructurally to enhance water quality. For example, Sacramento County is developing a Natural Streams Combining Zone for all property within the 100-year floodplain of county streams. By requiring permits or approvals for all uses in the floodplains, more natural stream courses and habitat will be preserved than were preserved under former planning processes. Government agencies should continue to enact rules to protect wetlands and riparian vegetation, particularly in the aftermath of Proposition 13, which greatly reduced available operation and maintenance funds.

Emergency measures to repair flood damage, by their nature, are rarely subject to an evaluation of their environmental impact, their consistency with state policy (such as protection of wetland and riparian habitat), or their cost effectiveness before they are undertaken. Nevertheless, analyses of these impacts and effects should guide both future emergency action and the long-term policies needed to prevent or reduce future damage. SB 366 provided funds for the Department of Water Resources to analyze the cost effectiveness of the emergency funds allocated to Los Angeles and Riverside counties and the City of Los Angeles. These analyses will provide us with guidelines in declaring future emergencies under the Water Code.

Government agencies in rural California are often reluctant to order or carry out prudent practices of nonstructural flood management. Yet rural areas are the very locations where floodplain regulation has the greatest potential to prevent flood damage and preserve environmental values.

The federal government should accelerate its studies in rural areas and start a program to inform residents of the rare opportunity they have to prevent the disasters and unnecessary cost suffered by some more developed areas. Rural administrators and the public could benefit from graphic illustrations of the flood damage that has struck more populous parts of California that failed to adopt nonstructural flood management practices.

Agencies that aggressively encourage public involvement in forming and planning flood programs and projects tend to experience fewer delays and objections from the public and generally achieve greater public acceptance of their actions.

All levels of government should realize that active and effective public participation early in flood management planning will make the public aware of the flood hazard, educate them about governmental concerns and constraints, and produce a willingness to help design a project acceptable to those directly affected. Public involvement is required in the processes of local approval, producing an environmental impact report, and right-of-way acquisition; it can also prevent delays, litigation, and rejection by decision-making bodies. Public involvement basically combines the needs and wishes of various publics (and there are many!) with the professionals' knowledge of a government agency to achieve a result that optimizes the efforts of both.

CONCLUSION

I have tried to show the need for improved cooperation among the federal, state, and local governments in flood management and to suggest where these improvements can be made. We need better rules and processes at all levels. Local governments must shoulder more of the burden of protecting the lives and property of their citizens; they can best do so by better planning before a crisis rather than by cleaning up after a disaster. Local flood control districts need to be involved in the process of planning and development approval.

The key to managing floods is to manage ourselves; we must all talk to each other and heed what the other is saying, particularly when the public speaks. The public is, after all, who we work for, and we must do our utmost to better shape bureaucratic processes to the democratic process.

APPENDIX: REDUCING FLOOD DAMAGE*

To explore ways of reducing flood damage, Los Angeles County drainage area was selected as an example of an urban area that suffered heavily in the disastrous storms of January and February 1980, even though it had an extensive flood control program. Most of the damages sustained by homes, roads, and other developments were caused by mudflows and erosion, resulting from overflowing streams, storm drains, and debris basins.

Under natural conditions the heavy runoff would have been carried away by the stream channels, and, if the water exceeded what they could handle, it would have overflowed and carved additional channels on the floodplain.

But, through the years, the capacity of the natural watercourses has been reduced by stream channelization, development encroaching upon the floodplains, the dumping of materials into the stream channels, the eroding of hillsides and river banks, and the paving over of land where water could once percolate into the ground.

Channels and storm drains have been built but do not replace this lost capacity. In normal times they may be sufficient to handle the runoff. But in heavy runoff the water or debris, when it overflows, cannot cut additional channels in the floodplain because they have already been built upon. Therefore it is forced to seek an outlet through streets, roads, houses, or whatever other structures stand in its path. Adding to the woes in localized areas are plugged drains and inlets, pump failures, and inadequate drain capacity.

Thus it is clear that, to reduce damages, we will have to:

- Develop and enforce adequate land use controls.
- Give priority to nonstructural floodplain management over structural solutions.
- Require that homes or other structures damaged more than 50 percent by floods meet new standards or be removed or relocated.
- Approve and accept drainage facilities only if they meet flood control district standards.
- Require that stability of hillside lots be certified by engineers and geologists before building permits are issued.
- Implement watershed management of hillside areas (erosion and runoff control by planting slopes, installing retarding basins, and other methods).

---

*This appendix is reprinted from the brochure Reducing Flood Damage, which was released by the Department of Water Resources in September 1980.

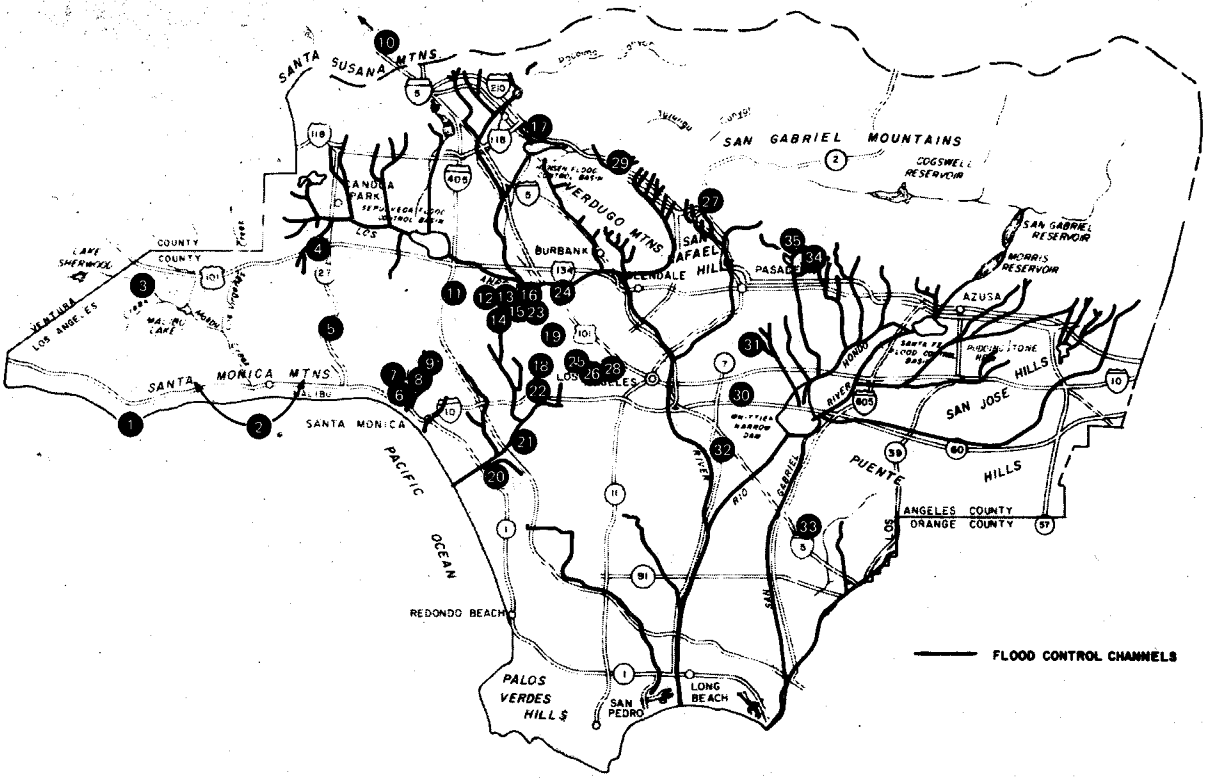


FIGURE 1 Identified with numbers in black dots on the map are representative areas in which flood damages were suffered in the storms of January and February 1980. The damages sustained at each of the locations identified on the map are listed in the table below. Beside each of these damages are one or more letters indicating the actions that need to be taken to prevent such damages in the future. These are keyed to a list of mitigation actions following the table.

Number on Map	January–February 1980 Damages	Mitigation Actions
1	Malibu. Debris washed to sea, plus heavy seas, battered pilings and bulkheads of homes on Pacific Coast Highway. Some invaded by mud and water from hillsides; sandbagging required.	B, H, J, M, T
2	Pacific Coast Highway. Closed from Santa Monica Freeway to Pt. Dume by mudslides and flooding.	C, G, M
3	Malibu Canyon. Mulholland Dr. closed from coast to Malibu Lake because road flooded or washed out. In Monte Nido woman killed when buried by mud.	F, G

4	Woodland Hills. Home near Ventura Freeway and Ave. San Luis inundated when Dry Canyon flood control channel overflowed.	G, I
5	Topanga Canyon Rd. Washed out in places.	O
6	Bel Air Estates. Swift current through intersection of Sunset Blvd. and Charring Rd.; people rescued from cars. Water 1.2 m (4 ft) deep at Sunset and Chautauqua.	G, I, P, Q
7	Rivas Canyon. Yards of three homes eroded when double-wire revetment washed out. Owners sandbagged threatened homes.	B, F, K, R
8	Mandeville Canyon Rd. Flooding and mudflows for 4 km (2.5 miles).	C, I, P, Q
9	Mandeville Canyon. Mudflows through homes forced 200 persons to evacuate; one house lost; one person killed by wall of flowing mud; 20 homes damaged by mud.	A, B, C, D, E, J, T
10	Saugus. Magic Mtn. Parkway closed for 0.5 km (0.3 mile) because washed out in places.	G, P, Q
11	Stone Canyon Rd. Mudslides closed at Valley Vista Blvd.	M, Q
12	Hollywood Hills. Mudslides blocked 13200 block of Mulholland Dr.	M, P, Q
13	Coldwater Canyon. Flooded on San Fernando Valley side.	G, I, P, Q
14	Trousdale Estates. Water and mud broke open doors and damaged living rooms of homes.	A, D, J, T
15	Laurel Canyon. One home demolished by mudslide; another collapsed when supporting ground became saturated and started creeping.	B, D, F, J, T
16	Laurel Canyon. Forty homes near Mt. Olympus evacuated, mainly because of mudflows, with erosion in yards.	A, B, C, D, E, J
17	Kagel Canyon. Four homes flooded when debris clogged drain, causing overflow.	B, G, I
18	Beverly Hills and West Hollywood. Gravel and mud on streets in localized areas; underground garage and foyer of apartment house flooded.	G, L
19	Hollywood. Residents on Genesee Ave. alerted for possible evacuation if Nicholas Canyon debris basin overflowed. (Basin did not overflow.)	I

20	Culver City. Flooding closed Lincoln Blvd. between Jefferson Blvd. and W. 83rd St.	G, I, P, Q
21	Baldwin Hills. Mudslides closed La Cienega Blvd. between Stocker St. and Rodeo Dr.	M, Q
22	Beverly Hills. Flooded at Olympic and La Cienega Blvds.	G, I
23	West Los Angeles. Laurel Canyon Blvd. flooded near Lookout Mountain Rd. One woman swept down to Hollywood Blvd.	G, I, P, S
24	Hollywood. Mudslides closed Cahuenga Blvd. between Pilgrimage Theatre and Barham St.	M, Q
25	West Los Angeles. Rossmore Ave. flooded between Beverly Blvd. and W. 3rd St.	G, I
26	Los Angeles. Underground garage on S. Kenmore St. flooded.	G, L
27	Angeles Crest Highway. Mudslides and washouts closed 3 km (2 miles) north of Foothill Blvd.	M, N
28	Los Angeles. Elevator shaft in Queen of Angels Hospital flooded; also basement and first floor of St. Vincent Medical Center and Midway Hospital.	G, L
29	Verdugo Hills Cemetery. Coffins exposed by erosion.	J, T
30	Monterey Park. Four houses collapsed and two knocked off foundation when mud slammed into them. Mudslides overtopped mud barrier and destroyed house.	B, C, D, F,J, M, N, T
31	Alhambra. Backyard of home eroded when concrete wall of San Pasqual Creek channel undermined and collapsed.	K, R
32	City of Commerce. Santa Ana Freeway flooded between Garfield andWashington Aves. Also, intersection of Atlantic Ave. and Telegraph Rd.	G, I, P, Q
33	Norwalk. Imperial Highway flooded near Bloomfield St.; people rescued from tops of cars.	G, I, P, Q
34	East Pasadena. Mud from hillsides closed several streets in Pasadena Glen. Man caught by mud and water swept 0.4 km (0.25 mile) downstream.	C, G, I, M, Q
35	Altadena. Rubio Diversion Channel became clogged and overflowed, carrying mud into 20 homes.	B, E, I

Actions to Prevent Future Damage

(Letters correspond to those under "Mitigation Action" in table.)

A. Floodproof homes and buildings with permanent walls or removable wood planks and architecturally pleasing deflector walls to guide flow to channel.

B. Implement strict land use controls to prevent structures from being rebuilt in areas of heavy damage and to prevent encroachment on streambeds. (For homes damaged more than 50 percent of market value, local government enforce requirements of National Flood Insurance Program for adequate protection from future damage; if protection cannot be provided, deny building permits.)

C. City, county, state, and federal land management agencies develop watershed management plan for mountain and hill areas to reduce amount of mud and debris carried down to developed areas. Should prohibit new bare cut-and-fill slopes, require runoff to be discharged into channels at low velocities, plant unstable slopes, and, in extreme cases, use structures to stabilize slopes and trap debris.

D. Strengthen and enforce ordinances requiring geologists and engineers to analyze stability of hillsides and flood control districts to approve proposed drainage facilities before issuing building permits.

E. Install esthetically pleasing deflecting and training walls at critical points along streams and at junctions to keep flows in channels and streets. Sandbags can be temporary measure.

F. Stabilize slopes next to buildings and critical sections of streets and channels. Example: homeowners use grouted riprap to transform channel into pleasing cascade. Terrace hillside slopes and install slope drains.

G. Check capacity of culverts, storm drains, and inlets, taking into account level of development in watershed.

H. Local government adopt and implement regulations restricting homes and development along shore within 100-year flood tide hazard boundary.

I. Develop and follow adequate maintenance program for drainage systems. Have crews available to clear debris basins between storms. Keep culvert and storm drains, particularly inlets, clear. Keep and analyze records to determine probable future problem areas.

J. Require in-depth study before issuing building and grading permits.

K. Reconstruct protective walls with deeper foundations to prevent failure by erosion.

L. Follow protective maintenance for private drains and sump pumps.

M. Use vegetative cover on slopes where control of mudslides not otherwise cost effective or practical. Covering slopes with plastic sheeting gives temporary protection.

N. Check adequacy of drains protecting cuts and fills.

O. Design outlets to prevent erosion with riprap or energy dissipaters.

P. Exercise strict traffic control during storms.

Q. Install signs on roads subject to flooding and mud and debris flows.

R. Construct grade control structures to blend into environment and to reduce flow velocity in channel.

S. If increased capacity of drain or channel is only feasible solution, assign priority for construction funds.

T. Relocation.

HAZARD MITIGATION: THE ENFORCEMENT TOOL

by Dale R. Peterson

The federal government has in the past responded to natural disasters such as floods and mudflows with response and recovery efforts that addressed the immediate need to get a community back on its feet and provide services to its citizens after the disaster. Unfortunately, in our desire to be responsive, unwise decisions have been made by all levels of government to allow unwise or unregulated development within known hazard areas.

The Federal Emergency Management Agency (FEMA) is now charged with the responsibility of mitigating future hazards by providing alternatives to the more traditional response and recovery efforts. The premise of FEMA's hazard mitigation programs is to provide a way to achieve long-range solutions. These programs, complemented by more effective local codes and development regulations, address what FEMA considers the most critical disaster recovery effort--the reduction of future losses of life and property.

Hazard mitigation measures were tested in fiscal year 1980 for the first time. Although FEMA believes that the programs are sound, constant refinement to improve development standards so that they are tailored to fit the communities' needs will be our challenge. All levels of government must support this effort so that our past development mistakes in flood and mudflow hazard areas will not be repeated.

At the outset I would like to clarify several misconceptions about the Federal Emergency Management Agency. This newly created executive-branch agency is a single point of contact in the federal government for emergency matters and coordinates the multiple use of resources in preparing for and responding to nationally declared emergencies.

Much has been said about problems with FEMA's responsiveness to the flood disasters of 1980. It must be understood that it is not FEMA's role to build a community back to its condition prior to a disaster; FEMA seeks rather to help all public and private sectors get back on their feet and mitigate future

---

Dale R. Peterson is Floodplain Management Specialist with the Federal Emergency Management Agency in San Francisco, California.

losses due to flooding, mudflows, or coastal inundation. FEMA has initiated several new programs to relocate to hazard-free sites structures repeatedly damaged by floods and mudflows. We have four such projects under way in California and Arizona, with expenditures totaling $3 million. These new programs, combined with established FEMA disaster programs that provide individual assistance to recoup personal property loss and that provide public assistance to restore overall public services such as bridges, roads, and sewage treatment facilities, expand on hazard mitigation programs and offer alternatives to local governments.

I represent the agency's Division of Insurance and Mitigation in Region IX. Formerly we were known as the Federal Insurance Administration under the Department of Housing and Urban Development, and we still administer the National Flood Insurance Program (NFIP). Our primary responsibility is to provide technical assistance through detailed flood insurance studies of hazardous areas. This is done so that communities can develop appropriate regulatory controls that will reduce future losses due to flooding or mudflows.

This year we have initiated a redirection of the program to address more firmly the philosophy of hazard mitigation. Supporting this objective are:

1. <u>The President's Executive Order No. 11988</u>, which requires all federal agencies to assess the impact of any proposed floodplain development and, if possible, identify flood-free locations or otherwise mitigate the potentials for loss due to flooding.

2. <u>Section 1362 of the Flood Disaster Protection Act of 1973</u>, which enables FEMA to purchase flood-damaged properties insured by the National Flood Insurance Program. This allows property owners to relocate to areas not prone to flooding and requires that damaged areas be rezoned for use as open space.

3. <u>The declaration of constructive total loss</u>, which enables FEMA to assist property owners financially who are insured by the NFIP to relocate to ares not prone to flooding. In order for the property to qualify, the community must prohibit the repair of the structure or allow the repair only at significantly increased cost. Both programs require that damaged property be deeded to the community and put to use as public open space.

These hazard mitigation measures were tested for the first time during fiscal year 1980. The programs are sound and provide FEMA with a problem-solving tool to reduce future losses due to flooding and mudflows. Existing enforcement tools, via regulatory controls, can only complement this new effort.

I am heartened that hazard mitigation has become a theme tying together the majority of papers included in these proceedings. However, I am also angered that together we continue to make the same mistakes of allowing unwise development in known hazard areas. In the name of expediency we have been known to do anything to justify the end, or as we have referred to it, the emergency or temporary need. My experience is that nothing is more permanent than a temporary structure or decision. Almost without fail these decisions come back to haunt us, when in the next year we must replace the structure or facility again because of flooding we knew could occur.

San Diego County has experienced the "disappearing bridge" phenomenon this past year. Structures inadequately designed to traverse a watercourse were further compromised when culverts were used instead of span construction or other design techniques tailored to site-specific needs. Likewise, the City of Phoenix loses not only bridges across the Salt River yearly but a sizable portion of the Sky Harbor Runway as well. The movable natural channel and the city's inability to stabilize the river's banks through the metropolitan area will continue to produce flood damages in the community. The costs to both the public and private sector continue to grow. Hazard mitigation through structural solutions will continue to cost the taxpayer more and more each year. One must evaluate the cost-benefit ratio of solutions to immediate needs in terms of the application of those solutions to long-range hazard mitigation.

The development of subdivisions in areas subject to mudflows and flooding from alluvial fans highlights the problem of adequate enforcement tools. The apparent inability of communities to learn from last year's mistakes is clearly evident in the Southland. Relaxed building codes and a willingness to issue variances to performance standards will continue to plague governments as long as the ostrich approach to regulating future development continues. The issuance of a variance because of "economic considerations" offends me. Yes, it will cost more to develop while recognizing an identified hazard. However, this additional cost is the price necessary to minimize future flood-related expense. How far will governments go to encourage risk development? We must awaken to the folly of these actions.

The Federal Emergency Management Agency continues to add to its experience with each event. The unique hazards in the Southwest range from riverine flooding to coastal inundation to alluvial fans to movable streambeds to mudflows; each is currently being addressed by FEMA. Before the National Research Council is a proposed methodology to study, identify, and map areas subject to inundation by mudflows. In addition, FEMA is this year initiating a physical modeling study to evaluate the nature of floodflows on alluvial fans and the associated problem of sediment transport. It is hoped that these studies will yield a method whereby improved and innovative development standards can be initiated. Papers included in these proceedings tell of the need to go beyond our current ability. In this regard, hazard mitigation is a viable planning tool. First, however, communities must recognize that there is a problem and be willing to address alternatives to mitigate the hazard. Hazard mitigation activities involve coordination efforts at all governmental levels, local, state, and federal. These efforts must be complemented by the enforcement of regulations on new developments and/or the replacement of existing developments after a disaster occurs.

The Federal Emergency Management Agency will not solve the problems that exist, but it can be used as a resource to address the need to mitigate future losses by wise management in hazard-prone areas. Hazard mitigation can and will be a solution to future flood and mudflow problems similar to those that occurred in California during 1980. It is our goal to provide the communities in this region with guidance to achieve this end.

THOUGHTS ON THE FLOODING DISASTER OF 1980

by Donald C. Tillman

During the storms of 1980 the City of Los Angeles experienced the worst rainfall on record. If one considers the entire storm series, it experienced possibly the greatest potential for disaster in many years. If not for the good disaster plans and operations in effect, and if not for many years of outstanding public works projects and field control, Los Angeles would have washed out to sea.

Whether it be for flooding or erosion, mudflows or landslides, fires or earthquakes, the City of Los Angeles is ready and is continuing to improve its interdepartmental operations through disaster exercises. Of particular concern in 1980 was the condition of debris basins within the city, which had been filling from the heavy storms of 1978 and the runoff from the barren areas of the 1979 fires. Reconnaissance and maintenance efforts were under way months before the rains of February.

There were two significant features of the rains that fell on the southern slopes of the Santa Monica Mountains. In Laurel Canyon, and probably in Nichols Canyon, a freak runoff was experienced, similar to that of 1978. A study is now under way to establish a special Disaster Alert for those canyons. It appears that a deluge of rainfall, combined with a confluence of peaks and narrow canyon topography, can result in a tidal wave of runoff. This occurred on February 14 in the early morning hours. On Saturday, February 16, 1980, the rain continued at a heavy intensity for about nine hours, without letting up. Never before have the San Fernando Valley and its drainage facilities been so taxed to capacity. The Los Angeles River, from the Sepulveda Dam across to North Hollywood, was above its capacity at many locations. Another hour of rainfall could have caused a super disaster.

The 1978 rains triggered many mudflows and landslides that purged the mountains and caused some of the buildup of geologic instability. However, the extent of the mud runoff in the valley on February 16, 1980, has never been equaled. Cleanup and repair activities have been costly. There is no accurate figure for damage to private property, but public facilities in the City of Los Angeles alone suffered $10 million of damage. No allowance was

---

Donald C. Tillman is City Engineer with the City of Los Angeles.

made for general pavement failures, such as the formation of potholes and the additional deterioration of paved surfaces, that could result. However, the city council provided $7.6 million, of which it is estimated that restoration costs are around $6 million. Federal reimbursement is now calculated at $2 million, though the amount of federal funds received as reimbursement to date is zero.

Future preparedness will depend largely on the funds available for maintenance between flooding disasters. The debris basins and channels must be kept clean to operate at full capacity. Storm drains and catch basins must be regularly maintained. In addition, there are still many missing links in an underground storm drain system that should be provided. We have recently estimated that the costs for all drains needed will total $600 million in the City of Los Angeles.

A disaster within the disaster has been federal reimbursement. The paperwork and numerous meetings held in the field during the design and construction operations of the recovery are excessive and unnecessary. The auditing and the attempt of local government to collect funds due from federal sources can go on for years. We are still trying to collect from the 1971 earthquake recovery work. Another fault I find with federal controls is that there are no national guidelines to prepare for decisions during the disaster and recovery period that influence eligibility for federal reimbursement. It makes a difference in the solutions taken if you must depend solely on nonexistent local funds. Standards of eligibility and guidelines for enforcement must be adopted now by the federal government for various types of disasters and must be given to local agencies for application. Otherwise, a total amount of federal money should be given to any area affected by disaster, to use as they determine proper, to make the necessary corrections. Auditing for uses on private property or illegal expenditures can then be performed afterward. This would minimize the number of meetings and expedite the repairs that the public needs. Some revisions are also needed in the federal government's procedures for Small Business Administration loans. The few citizens who are severely hurt by disaster must wait several months for the first loan funds to arrive and be of any assistance.

In summary, flooding in southern California is being controlled, but it is still sufficient enough to cause concern. Public officials, who face the loss of limited tax dollars, must protect the general welfare and safety of the public. The best approach for the future is to continue public works construction where critically needed, develop disaster manuals that will provide guidelines for all types of disasters, disseminate federal eligibility rules and regulations for reimbursement, and shorten the time it takes to process matters of management and administration so that they do not overlap from disaster to disaster.

FLOOD ASSISTANCE ON PRIVATE PROPERTY: PRIVATE RESPONSE TO A GOVERNMENT RESPONSIBILITY?

by Andrew Lipkis, Sherna Hough, and Lisa N. Geller

For the past three years the California Conservation Project, also known as TreePeople, has been called on to provide emergency assistance for a variety of flood-related problems on private property. TreePeople is a private nonprofit organization based in Los Angeles and involved primarily in reforestation and environmental education. The organization has a small paid staff supported by a large corps of volunteers. At the request of government agencies unable to respond to emergencies that arose from flooding in the Los Angeles area, TreePeople mobilized and dispatched up to 3,000 volunteers to participate in sandbagging, bridge building, and other immediate solutions to problems caused by extensive rains. Private citizens seeking help from agencies such as fire departments, flood control districts, and the police were referred to TreePeople for assistance in protecting their homes and mitigating damage. This is because of limitations on those agencies in terms of resources as well as limitations imposed by law. Volunteers were recruited from preexisting files of TreePeople and through the media. Because much of the damage is the result of runoff from fire-devastated hillsides, TreePeople rehabilitated burned slopes and offered preventative advice.

Citizens were often shocked and angered to learn that there is no government agency authorized to provide aid on private property. The question of whether the responsibility for emergency flood assistance to private individuals lies within government agencies or the private sector is as yet unresolved. In the interim a program is being developed by TreePeople to train volunteers who will be able to consult with homeowners on preventative measures and lead crews of volunteers during emergencies. Because the organization has been able to offer services without great expense or extensive new bureaucracies, this plan could provide an attractive model for governmental or private service organizations.

TREEPEOPLE EMERGENCY CENTER*

In February 1980 the TreePeople staff was on duty 24 hours a day for 10

Andrew Lipkis, Sherna Hough, and Lisa N. Geller are with the California Conservation Project (TreePeople) in Beverly Hills, California. Andrew Lipkis heads the organization.

*The rest of this report is reprinted from <u>TreePeople, Special Report</u>, April 1980, and was written by Sherna Hough.

days in a row. During that time they organized about 3,000 volunteers and saved nearly 1,200 homes from damage by a severe storm that was hitting southern California. At the peak of the storm--Saturday, Sunday, and Monday, February 16-18--as many as 150 crews were out at any one time, with crews ranging in size from 3 to 50 people.

These volunteer crews worked in cold wet conditions to help people they had never seen before, people they probably would never see again. They set aside all thoughts of "daily routine" to deal with the intensity of the storm. All problems were urgent, all working conditions uncomfortable. Sometimes they were so cold they couldn't believe they would ever get warm again. Sometimes they were so tired they didn't have the energy to drive home to go to bed. And yet, if another call came in and they were needed, they would go out again and again to help. Their reward was a free cup of coffee and a sandwich.

The TreePeople office was the scene of barely controlled chaos. When the Los Angeles Office of Civil Defense made TreePeople Headquarters the volunteer center for the entire city, Pacific Telephone had installed three emergency telephone lines into the Coldwater facility. Those lines, plus the office's normal four business lines, rang 24 hours a day for 10 days straight. All other activities that TreePeople normally are involved in had to be set aside for a time.

The activities of the TreePeople had to be stopped, but the philosophy of the TreePeople was in full swing. Andy Lipkis, Director of the TreePeople, had always believed that if people got together and worked together they could create miracles. He had normally applied that philosophy to environmental problems, such as smog and energy sources. But citizens' responses during the flood crisis proved the point in a way that no speeches could.

TreePeople unwittingly got into the "emergency business" in 1978, when three staff members went to help a neighbor whose home was being threatened by a landslide. As they had organized and equipped volunteers to plant trees in the past, they organized and equipped about 50 neighbors to help save that house. By chance, that night a representative of Los Angeles City Councilman Joel Wach's office was there. A few days later, when a storm was expected to hit the Los Angeles area, she called the TreePeople to ask if they would be able to organize volunteers if things got really bad.

The TreePeople had never done anything like that before, but they said they would try, and when the storm hit they mobilized about 800 volunteers to save nearly 300 homes over a period of three days. Since then, every time there has been a storm or fire in the Los Angeles area, some people have called the TreePeople asking for help in their homes, while other people have called offering to help. In each instance the TreePeople staff has done its best to connect the two groups of people.

The floods in February 1980 were not a surprise to flood control experts. The month before it had rained very hard for several days in a row. The southern California soil was saturated with water and simply couldn't absorb any more. In fact the TreePeople had gotten calls for help in January. When

The heroes of the operation were the private citizens, volunteers who worked in cold wet conditions to help people they had never seen before, people they probably would never see again. (Photographs by Jeff Share.)

the new rains started in February, the excess weight of the water on the slopes in mountain areas caused topsoil to begin to slide. Homes, cars, or swimming pools in the path of the slide were filled with mud and debris, or washed away.

Because of the magnitude of the storm, government workers were rendered helpless to protect private property. The police and fire departments worked very hard--and well--to protect public property, but there simply were not enough of them to go around. Their first priority had to be to help save lives. The protection of private property had to be left for private citizens to do. This is where the TreePeople came in.

It was Thursday, February 14, when Mike Regan, the city's Director of Civil Defense, visited TreePeople Headquarters at Coldwater Canyon Environmental Education Center. Knowing about their past experience with emergency work, he said that he would like the TreePeople to coordinate all volunteer activity in Los Angeles during storms that were expected to hit soon. He called the phone company to get extra phones put in, then gave the staff the phone number of a warehouse to call to get some cots and the number of the Salvation Army, which set up a disaster kitchen at Coldwater for the duration of the emergency.

The TreePeople made all of these arrangements, then called the media to send out a request for volunteers. Some people showed up that night, February 14, but it wasn't until Friday, February 15, that the storm really got bad. By Saturday, February 16, the storm was in full force, and the TreePeople operation was in full swing until Sunday, February 24.

TreePeople staff and volunteers answered the phones by saying, "TreePeople Emergency Resources Center." The people on the other end of the line were almost always frantic. Typically, they had called the police or fire departments first, to be told that there was nothing the city services could do for them, and were given the TreePeople number. The callers were shocked, frightened, and in urgent need of help. For many days TreePeople Headquarters was the only place they could turn to.

It was the job of the person answering the TreePeople hotline to calm the caller down and to confirm that nobody's life was in danger. If a person had to be rescued, the call was immediately connected with the fire department, which did have the resources to save human life. But if the problem was one of property damage, the TreePeople handled it.

The most difficult job the TreePeople operators had was to determine how much help the homeowner needed. Could 3 people do the job, or would 30 be needed? Would 5 sandbags take care of the problem, or would 500 still not be enough? Often this never really was clarified. Simply hearing the person on the other end of the line was a problem in the din of the office. But everyone did the best they could. Then they wrote down the information, plus the caller's name, address, and phone number, in specially printed dispatch sheets.

The dispatch center was in one of the new environmental classrooms at Coldwater, in what used to be a garage for fire engines when Coldwater was a fire station. The large room had no heat, few lights, and very little furniture. Volunteers sat on piles of sandbags, piles of sand, or on a dry spot on the floor if they could find one. They waited for a four-wheel-drive vehicle to be available to take them to a home that needed saving.

The four-wheel-drive vehicles were a key to the whole operation. At first almost all of the vehicles available belonged to members of a club called the Associated Blazers of California. For most of the club members their four-wheel-drive vehicles were recreational transportation, but they, as a group, had decided to use them for service activities as well as for fun. They had sponsored such events as ecology cleanup parties, and they had helped the TreePeople with tree plantings in the past.

When the TreePeople had agreed to organize their first emergency services during the flooding in 1978, the Associated Blazers had helped to make it possible. Now here they were again. For 10 days and nights they risked their personal safety and damage to their vehicles to take volunteers to places where cars could not go during the flooding. Many other people came to volunteer their four-wheel-drive vehicles during the crisis, but all transportation was coordinated by members of the Associated Blazers.

The other key to the operation was the WB6BJM Repeater Group, a club of ham operators. Like the four-wheel-drive vehicle people, the radio people owned their equipment primarily for fun, but they had agreed to train themselves to help in emergency situations. During the floods they set up a command post in the environmental library at Coldwater and sent a ham radio operator with as many of the four-wheel-drive vehicle teams as possible. This ability to communicate with volunteer teams all over the Los Angeles area was vital to the whole operation.

The combination of the four-wheel-drive vehicles and the communications system supplied by the WB6BJM Repeater Group was so effective that the Los Angeles Police Department asked these TreePeople volunteers to help them for several days. The four-wheel-drive vehicles could go up flooded streets that rendered the police cars helpless, and the ham radio equipment was far superior to the police radios for transmitting in the mountain passes.

It is impossible to assess exactly how much difference TreePeople's involvement in the disaster work made. However, it is clear that without their help things would have been much worse. In addition to the 1,200 homes that were directly helped, TreePeople volunteers covered large areas of hillsides with plastic, donated by a subsidiary of ARCO, to prevent further mudslides. They also pulled countless cars out from under piles of mud and helped keep police informed about the conditions of the roads.

How much is this worth? TreePeople spent about $18,000 in staff time, food, telephone bills, and materials. In return they received about $3,000 in donations to help pay for their costs. When the City of Los Angeles asked them to coordinate all volunteer efforts in the city, they did not say they would pay for the costs of doing this.

Despite the financial burden, the TreePeople would certainly respond to an emergency again. It is part of their philosophy: to take care of our environment includes taking care of the people in it. Furthermore, between storm-related crises the staff is eager to talk with homeowner groups about how to plant vegetation on hillsides to lessen the chance of mudslides during the next storm.

When the crisis is over it almost immediately becomes hard to believe that it happened. As soon as people are dry, warm, and rested it becomes hard to remember what it felt like to strain physical limits only a few hours before. As soon as the streets are dry and clean it becomes difficult to imagine them as rivers of mud. Yet the experience of citizens creating miracles will enrich those involved forever. And it becomes time for TreePeople to get back to doing what they normally do: planting trees to help ease the smog problems in the Los Angeles area, and educating people about the environment so they can perform greater miracles in the future.

CLOSING COMMENTS ON DEBRIS AND SEDIMENT

by John M. Tettemer

INTRODUCTION

The words "debris" and "sediment" have been used over and over again in the papers included in these proceedings. It is interesting, because the war stories we tell each other are almost exclusively related to debris problems, sediment problems, or the upsets caused by them. Maybe the primary conclusion is that debris and sediment are major enemies to flood control systems and that more attention should be paid to understanding them.

NEED FOR NEW POLICIES AND APPROACHES

I would like to suggest some new policies and approaches to the major challenges ahead. I think we can look back at the last 10 to 15 years and say that with the help of environmental interests, short money supplies, uptight communities, and changing priorities, most of us have learned that a wide range of flood control options are available besides concrete channels. I think most of us understand that natural values should be preserved, and many of us are successfully implementing floodplain management. While we may have been goaded by local communities, local groups, the California Department of Water Resources, the National Flood Insurance Program, and common sense, we still have come a long way and we can be proud of our work.

One question, however, has been left unanswered. It is the one that deals with "non-clear water" engineering. We have few problems with engineered channels so long as the water stays clear. But we have experienced enormous damage and frightening failures throughout southern California during 1977-80, all of which were related to erosion, sediment, and debris. In his paper "Presidentially Declared Major Disaster for Seven Southern California Counties--The January, February, and March 1980 Storms," Jacob Angel lists the total costs to public and private nonprofit facilities caused by storm damages during 1980. At the top of his list is removal of sediment and debris. This should be telling us something. Furthermore, I believe that the cost of sediment management will increase because of actions being taken by local governments (meaning you and me).

John M. Tettemer is Acting Chief Deputy Engineer with the Los Angeles County Flood Control District in Los Angeles, California.

As sediment moves from the mountains to the ocean through rivers, floodplains, and flood control facilities, it is viewed in many ways. Some of us want it to replenish the beaches, some do not want it in the harbors, some want it to stay in the mountains so it does not bother the floodplains, and everyone wants it someplace else. The only thing for sure is that it has to be dealt with as long as people live between the mountains and the ocean. We need innovative new management strategies that are based on a sound understanding of sediment engineering and that are applicable in a range of economic environments.

EXAMPLES OF PROBLEMS

Let me share a few problems of Los Angeles County to set the stage for a proposal.

The Santa Clara River drains about 800 square miles of Los Angeles County to our neighbors in the west. It supports a wide variety of land and activities: dense urbanization, rural and pastoral settings, deserts, manufacturing, and the Magic Mountain amusement park. In this allotment of use there are competing needs and problems. For example, if we managed sediment in the Santa Clara River system the way we have been managing it in the Los Angeles basin, we would end up with a sediment starvation problem. We have seen that our neighbors in Orange County face very serious sediment starvation problems. When we trap the sediment, levee linings will be undercut, as were those in Ventura County and Santa Barbara County. On the other hand, if we allow the sediment to come into the system, we end up with accretion of riverbeds. In reality, we do not know what the sediment transport balance is in the Santa Clara system. We do not know what the 50-year cycles are. We need to know and we want to know; if we do not understand the system we will have to spend millions of dollars to stop erosion by retrofitting channels with stabilizers, as they are doing in Santa Barbara County, or by driving the toe rock down another 20 ft. We are not willing to spend that kind of money based on the present engineering state of the art. We would prefer to avoid expensive adjustment by pursuing strategies that are based on understanding the mechanics of the river.

There are some other problems. Funny things are occurring in small debris basins. I cannot tell you exactly what. All I know is that during major storms, debris sometimes goes over the spillway when the basin is not really full. We do not understand the dynamics, but it seems that these small basins act more like flip buckets than lakes. The model of the placid lake quietly filling with mud does not seem to fit. I do not have the answer, nor do I think any of us have it. We need to know, however, because this problem affects the public's safety below these facilities.

Another problem is literally in our own backyard: the small backyard canyon we so often overlook. It is very easy to recognize sediment and debris problems in a big canyon. However, we are all getting blitzed after fires by small canyons. Here is where we need to review proposed developments lot by lot to evaluate the debris hazard from a fire and flood sequence. We can then design lots, street patterns, and building sites with full recognition and accommodation of the hazard.

Another sediment-related problem is the abrasion and destruction of concrete channel bottoms by sediment-laden flow. We and the U.S. Army Corps of Engineers expected our channels to last maybe 100 years. We have seen cases where the reinforcing steel was exposed in one storm. Almost no research has been done on this subject. We are installing scour gages in the channels to monitor wear and gather data.

In summary, I am trying to emphasize that we really do not have all the answers that we think we have concerning sediment, whether it is in a river, a concrete channel, or a small debris basin or is generating the enormous sand waves we have heard about. By the way, I saw some sand waves that were 8 ft high standing in the middle of nowhere in Little Tujunga Creek. I do not understand how they got there, nor do I understand how they could be predicted in a manner that would influence our design. It is another question we need to answer.

A REGIONAL CHALLENGE

These problems throughout southern California, Nevada, Arizona, and New Mexico are in areas of "hard to develop" land. It may be hard to develop because it is part of an alluvial cone that we do not fully understand or have not completely planned out. It may be a coastal plain at the interface between the ocean and a sediment-carrying stream. It may be a steep canyon high above the lights in the valley below. Even though we are diligent as planners and regulators with clear water, we may not be diligent with sediment and debris. We do not know enough. We are getting involved in enormously increased governmental expense and costly retrofitting. Both of those can be avoided through good planning. The problems have generated concern among our political bosses and at the state and federal levels. We also find communities starting to wonder why their high-priced facilities fail during storms when they are needed. I do not blame them, do you?

PERSPECTIVES

During the last several years I have had an opportunity to work very closely with the entire hierarchy of flood watchers. From the policy maker's standpoint there are several interesting perspectives. He or she must consider the feeling of the homeowner who says, "Why was this house built here? Why was it allowed to happen? Why didn't that backyard drainage facility work? Why were we flooded?" These are good questions. Can we answer them? I cannot. The houses should not have been there. In most cases they are safe from clear water flows but they are not safe from sediment or debris.

The home builder has another perspective. He strives for minimum cost and operates on both the engineering and the political levels. If we keep his costs down, everything is cool. His objective is to sell the product and move along.

The third perspective is that of local government, which inherits the problems. We are paying dearly throughout the Southwest for those problems. Our attitude in local government is mixed. Some of us have the courage to deal with the situation; others do not. There is apathy. There are mixed signals. Some of us are saying that we have the right idea but that we cannot sell it to our political bosses, or cannot sell it to planning, or cannot sell it to the community. These are difficult problems that need to be tackled. To convince others about sediment and debris hazards, we need to know more. As an industry of engineers and scientists, we need to gang up and have overwhelming information on sediment. Then we can get a better toehold into the politics.

The final perspective is that of the federal government, an interesting animal. It loves to bail people out, and we have become experts in ringing the chimes that bring in the federal disaster funds. All the while we go happily along replacing last year's problems without mitigation. We owe a debt of gratitude to the Federal Emergency Management Agency (FEMA). The National Flood Insurance Program is a giant step in the right direction. We should stop building things where they do not belong and leave some room for nature. FEMA had the courage through their political mechanizations to try to pull a nation together on obvious storm damage problems. They have done a good thing. I have been involved in implementing the flood insurance program in many cases. If you are careful and smart and if you stay ahead of your political bosses, you can show the benefits to them and to the community of handling federal flood insurance in the right way. We have managed, luckily maybe, to convince some die-hard antigovernmental people that the program is right and they should support it. Now some of them will go out and take on large groups in community meetings, supporting the National Flood Insurance Program. I am not saying it can be done everywhere, but it is the best thing that has happened to state and local government in the interest of safe development. What they have done is to tell us to apply some common sense and knock off the business of creating problems for the pleasure of solving them.

UNDERSTANDING AND APPLICATION--A COMMITMENT FOR THE 1980s

I would suggest that we look forward to the 1980s and to a new challenge that I would leave with all of us. The 1980s must be a period of recommitment to basic engineering and to the application of that engineering to our projects and regulatory functions. If sediment and debris problems represent the major cause of failure or upset in our systems, we should commit ourselves to greater understanding and wiser application of what we know. Floodplain management and other soft or nonstructural solutions will be relied upon by the public just as much as the concrete channels. They must be every bit as reliable. We have seen that they are not yet fully reliable.

ACTION PLAN

Many of us feel that we understand the engineering principles involved but that community and budgetary problems are consuming all of our energy. Maybe so. The storms of the last three years tell me that we can accept neither our present level of understanding or the obstacles. I would recommend that we recommit ourselves through a four-step program.

1. Go back to our organizations and examine them. What is their understanding of the engineering? What do they really know? What needs to be

learned? Who are those most capable of pursuing the necessary knowledge? What field data exist?

2. Collectively as a group, or in professional associations, pool data, knowledge, and resources to develop standards for some of the things we are talking about. Get together at meetings and exchange ideas and come up with a basis on which the Southwest can build and regulate itself.

3. Join with the Los Angeles County Flood Control District in urging the federal flood insurance people to proceed with the mapping program for so-called mudslide hazards. They have contracted with the National Research Council to review and assess methodology for mudslide mapping. I would encourage all of you to prod all of them, at both the political and engineering levels, to get on with it. I would not say that they are malingering, but after all these years of flood mapping we still do not have maps that show the true sediment hazard. You need them, I need them, we all need them. We should let the public know what is going on in these communities. I think it will change your own staff's view of what is safe and what is not safe. We need the maps and we need to push for an appropriation to get them started.

4. As we are working on the first three steps, I suggest that we work together to develop a new understanding of "sediment engineering practice in local government." This can be used when the opportunities arise to influence planners, zoners, and building people. It will help them understand the problem and the solutions. In my view the potential for sediment damage is getting worse. Not enough energy is being put into it. It is the sediment and debris aspects of drainage that create the upsets, and we, who are presumably the leaders in this field, bear a grave responsibility to solve the problem before it gets completely out of hand.